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ADVANCED SOIL SAMPLING OF SILTY SANDS IN CHRISTCHURCH

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Executive Summary

Current research in geotechnical engineering at the University of Canterbury includes a number of laboratory testing programmes focussed on understanding the behaviour of natural soil deposits in Christchurch during the 2010-2011 Canterbury Earthquake Sequence.

Many soils found in Christchurch are sands or silty sands with little to no plasticity, making them very difficult to sample using established methods. The gel-push sampling methodology, developed by Kiso-Jiban Consultants in Japan, was developed to address some of the deficiencies of existing sampling techniques and has been deployed on two projects in Christchurch.

Gel push sampling is carried out with a range of samplers which are modified versions of existing technology, and the University of Canterbury has acquired three versions of the tools (GP-S, GP-Tr, GP-D). Soil samples are extracted from the bottom of a freshly drilled borehole and are captured within a liner barrel, close to 1m in length. A lubricating polymer gel coats the outside of the soil sample as it enters the liner barrel. The frictional rubbing which normally occurs on the sides of the soil samples using existing techniques is eliminated by the presence of the polymer gel.

The operation of the gel-push samplers is significantly more complicated than conventional push-tube samplers, and in the initial trials a number of operational difficulties were encountered, requiring changes to the sampling procedures. Despite these issues, a number of high quality soil samples were obtained on both projects using the GP-S sampler to capture silty soil.

Attempts were made to obtain clean sands using a different gel-push sampler (GP-TR) in the Red Zone. The laboratory testing of these sands indicated that they were being significantly disturbed during the sampling and/or transportation procedures.

While it remains too early to draw definitive conclusions regarding the performance of the gel-push samplers, the methodology has provided some promising results. Further trialling of the tools are required to refine operating procedures understand the full range of soil conditions which can be successfully sampled using the tools.

In parallel with the gel-push trials, a *Dames and Moore* fixed-piston sampler has been used by our research partners from Berkeley to obtain soil samples at a number of sites within Christchurch. This sampler features relatively short (50cm), thin-walled liner barrels which is advanced into the ground under the action of hydraulic pressure. By reducing the overall length of the soil being captured, the disturbance to the soil as it enters the liner barrel is significantly reduced.

The *Dames and Moore* sampler is significantly easier to operate than the gel-push sampler, and past experience has shown it to be successful in soft, plastic materials (i.e. clays and silty clays). The cyclic resistance of one silty clay obtained using both the gel-push and *Dames & Moore* samplers has been found to be very similar, and ongoing research aims to establish whether similar results are obtained for different soil types, including silty materials and clean sands.

Contents

Executive Summary 2

Acknowledgements 5

1. Introduction 6

  1.1 Scope of study 6

  1.2 Report Outline 7

2. Advanced soil sampling 8

  2.1 The case for undisturbed sampling and testing 8

  2.2 Advanced sampling techniques 9

    2.2.1 Conventional fixed-piston drive sampling 10

    2.2.2 Rotary samplers 11

    2.2.3 Other Methods 11

    2.2.4 Introducing gel-push sampling 12

  2.3 Samplers trialled in this study 13

    2.3.1 Gel-Push piston sampler (GP-S) 13

    2.3.2 Gel-push triple tube sampler (GP-TR) 14

    2.3.3 The Dames and Moore piston sampler 15

  2.4 Methodology 16

    2.4.1 GP-S Sampler 16

    2.4.2 GP-TR sampling 18

    2.4.3 Dames and Moore sampling 20

  2.5 Preserving sample quality after sampling 21

    2.5.1 Onsite 21

    2.5.2 Transportation 21

    2.5.3 Laboratory handling 21

    2.5.4 Dames and Moore sample handling 23

3. Sample quality evaluation 24

4. Testing procedures for undisturbed samples 27

  4.1 Saturation and consolidation 27

  4.2 Bender element testing 27

  4.3 Cyclic triaxial testing 28

  4.4 Measurement of volumetric strain after liquefaction 29

  4.5 Particle size distributions 29

5. Advanced Sampling in Christchurch 30

  5.1 Projects conducted 30

5.2 Field Trials in the Christchurch CBD 30
5.2.1 Sampling sites 30
5.2.2 Sampling operation 31
5.2.3 Materials encountered 34
5.2.4 Quality of samples retrieved 35
5.2.5 The Characterisation of Christchurch Sands for Modelling.. 41
5.2.6 Key findings in relation to GP Sampling from the Initial GP sampler trials 42
5.3 The Silty Soils Project.. 44
5.3.1 Site selection 44
5.3.2 Characteristics & targeted sampling profile.. 46
5.3.3 Sampling Performance at Gainsborough Reserve.. 48
5.3.4 Liquefaction resistance of Gainsborough Reserve soils 49
5.3.5 Key Findings.. 52
6. Summary and Conclusions.. 53
References.. 54
Appendix A.. 60
A1: Sample tube geometry indices. 60
Area Ratio and Cutting Edge Taper Angle: 61

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1. Introduction

This report presents a summary of the recent geotechnical engineering research undertaken at the University of Canterbury to sample, test and characterize the behaviour of Christchurch silty sands, using undisturbed samples recovered from several sites in Christchurch. The work includes the acquisition, trial and use of new *Gel-push* soil samplers of both fixed-piston and rotary designs, developed jointly in Japan and Taiwan. The sampling and testing of undisturbed samples of Christchurch sandy soils builds on recent laboratory-based testing and characterisation work undertaken at the University of Canterbury, specifically the work on undrained monotonic and cyclic response of silty sands by Rees (2010) and the dynamic soil behaviour of silty sands by Arefi (2014). Key aspects of interest necessitating undisturbed sampling include the natural features of soil deposits such as ageing (creep, stress-history, and diagenetic processes), fabric, soil-structure (inhomogeneity in soil grain size and density), not captured by laboratory based studies on reconstituted specimens.

The aforementioned studies on liquefaction characterization of Christchurch soils (Rees, Arefi) were initiated before the Canterbury earthquakes, in 2006. Their particular focus was on the effects of fines on the liquefaction resistance and cyclic stress-strain behaviour of sandy soils. The widespread liquefaction triggered in the 2010-2011 Canterbury Earthquake Sequence (CES) sharpened the focus of our studies and provided abundant field evidence on the performance of Christchurch silty sands under severe earthquake loading and multiple earthquake events. Significant field-based penetration testing has been conducted across Christchurch (more than 18,000 CPT collated on the Canterbury Geotechnical Database), allowing for the evaluation of liquefaction triggering hazard through use of the semi-empirical 'simplified' method, i.e. the Seed and Idriss (1971) approach and recent derivatives such as Boulanger and Idriss (2014) and Robertson (2009a). The performance of various simplified procedures have been scrutinised by several studies (e.g. Bray et al., 2014; Taylor, 2015; Green et al., 2014; and Tonkin and Taylor, 2013) showing consistent mis-predictions and conservatism in the predictions. The possible reasons for these outcomes are many and are worth exploring in further detail at specific sites to better understand the limitations of the simplified method and areas required for refinement. Some sites observed across the city exhibited no effects of liquefaction (e.g. sand boils, ground failure) despite predictions of significant liquefaction triggering through application of the simplified method. A means to directly evaluate a soils cyclic response and advance our understanding of fundamental behaviour of soils is via undisturbed sampling and laboratory testing under controlled conditions. As sandy soil samples may be readily damaged by the sampling process, advanced techniques are required that are both technically successful and cost effective, for both sampling and laboratory testing. These studies make a pioneering contribution in New Zealand using such efforts and also leading contribution internationally on the subject.

1.1 Scope of study

The sampling of soils below the water table is non-trivial and new techniques such as Gel-push (GP) presented herein are no exception. Research effort to understand and develop experience with the new technique, to optimise the drilling and sampling method for the given soil conditions in Christchurch, and in time further afield within New Zealand, is required. This report presents a first effort to develop and document this experience in application to selected sites of interest across Christchurch. Some specific questions relating to the undrained cyclic behaviour of undisturbed samples were considered through the initial trials of the GP samplers in the Christchurch CBD, and the evaluation of the response of Christchurch silty soils at the sites where liquefaction predictions were poor provided a driver for further use of the samplers. Some aspects and outputs from these projects are summarised in this report.

1.2 Report Outline

This research report documents the drivers for undisturbed soil sampling, the methods that have been developed internationally to sample sandy soils to a high quality, their advantages and disadvantages. The new Gel-push samplers obtained from Japan are introduced, alongside the *Dames and Moore* Piston sampler obtained from the US, with research partners at the University of California, Berkeley. A summary and guideline for Gel-Push sampling is also provided. Methods to test the undisturbed samples in the laboratory and assess the quality of the specimens are also presented. Two research project applications are also outlined:

- The initial GP sampler trials in the Christchurch CBD, August 2011
- The Silty Soils Project conducted at specific sites across Christchurch 2013-2015.

Initial findings, recommendations and future research directions are presented and briefly discussed.

2. Advanced soil sampling

2.1 The case for undisturbed sampling and testing

Advanced soil sampling, that provides practically ‘undisturbed’ samples for laboratory testing, is recognised as an important technique to aid in reducing uncertainty associated with empirical correlations routinely used in engineering practice, and to improve the scientific understanding of the behaviour of soils with specific attributes. It is seen as an essential part of a comprehensive geotechnical site characterisation, that draws on both the advantages of *in situ* testing for profiling of the soils at a site, and laboratory testing of soils under an environment closely mimicking in situ stresses, drainage, and loading conditions, while using undisturbed samples (refer Table 1 for the general pros and cons of laboratory and field testing methods after Ladd & DeGroot (2003) and Clayton and Hight (2006)).

The cyclic behaviour and liquefaction resistance of a given soil has been shown to be significantly affected by a soil’s *state parameters*: density, confining stress, soil macro-structure (e.g. layering and laminations, macro variations in density), and soil micro-structure including fabric (arrangement of particles) and bonding (e.g. cementation) (Ishihara, 1993). While the effects of *in situ* density and confining stress may be approximated in the laboratory with reconstituted specimens, the influences of the natural soil structure (arrangement of particles and layering at micro scale) are impossible to reproduce exactly in the laboratory. Ageing effects such as secondary consolidation, stress-history and chemical bonding/ cementation of particles are also highly difficult to reproduce in the laboratory, but are known to have a significant impact on the cyclic response of soils and their behaviour during earthquakes (Youd and Perkins, 1978; Mitchell and Solymar, 1984; Ishihara, 1993; Tokimatsu et al., 1986; Andrus et al., 2009). In view of the importance of soil fabric and time effects, obtaining undisturbed samples is desirable in order to measure the effect of both *in situ* density and soil structure, including grain-to-grain contacts, on the cyclic response of soils. Hence, currently two approaches to liquefaction hazard assessment are available: testing of high quality undisturbed samples obtained from the field (e.g. Ishihara et al., 1978; Yoshimi et al., 1994), or using the `simplified method’ for liquefaction evaluation which relies on empirical correlations between field testing parameters and liquefaction resistance developed from case histories (e.g. Boulanger and Idriss, 2014; Seed et al., 2003; Robertson, 2009a; Youd et al., 2001; Kayen et al. 2013). The latter approach assumes that penetration resistance of soils (from CPT or SPT test) to some degree accounts indirectly for the complex state and ageing effects on soil behaviour and liquefaction resistance. Each approach has advantages and disadvantages, but for characterising soils other than ‘conventional’ clean sands that dominate the case-history database (e.g. sands with significant fines, including the influence of soil plasticity; aged soils; crushable sands; gravelly soils), advanced sampling and laboratory testing offers some clear advantages for an in-depth understanding of soil behaviour and liquefaction hazard evaluation. Soil characterisation informed by advanced sampling and testing may also be readily used to calibrate soil models for advanced seismic analyses which are capable of simulating complex liquefaction processes and allow for assessment of the effectiveness of various countermeasures (e.g. ground improvement methods) against liquefaction (Cubrinovski, 2011).

Table 1: Pros and cons of in situ and laboratory testing for soil profiling and engineering properties (from Ladd & DeGroot, 2003)

	In situ testing e.g SCPTu, SDMT,	Laboratory testing on undisturbed samples
Pros	Best for soil profiling: • More economical and less time consuming • (Semi) continuous record of data • Response of larger soil mass in its natural environment	Best for engineering properties: • Well defined stress-strain boundary conditions • Controlled drainage & stress conditions • Known soil type and microfabric
Cons	Requires Empirical Correlations for Engineering Properties: • Poorly defined stress-strain boundary conditions • Cannot control drainage conditions • Unknown effects of installation disturbance and very fast rate of testing.	Poor for soil profiling: • Expensive and time consuming • Small, discontinuous test specimens • Unavoidable stress relief and variable degrees of sample disturbance

2.2 Advanced sampling techniques

Over the years, a number of different sampling techniques have been developed, ranging from relatively simple methods such as block sampling and push-tube techniques, to more complex methods. The more advanced sampling methods include hydraulically activated push tubes, rotary sampling and freeze sampling. The costs associated with undisturbed sampling range from relatively inexpensive (i.e. conventional push tubes) through to being prohibitive for all but the largest projects (i.e. freeze sampling).

While results from a study based on undisturbed sampling can significantly reduce the uncertainty in soil characterisation, it is recognised that if proper care is not taken, or an inappropriate sampling technique is selected, the results from such studies can be of little or no value.

Clean sands have been shown to be particularly difficult to sample, due to:

- Volume changes during sampling resulting in compression or dilation of the sample caused by the high friction mobilised between sample and internal tube walls, or soil relaxation allowed to occur within a tube with larger diameter than the cut sample;
- Retrieval and associated changes in effective stress, which in turn result in deformation of the sample;
- Transportation and preparation for testing in the laboratory (vibration under low effective stress) requiring special care to avoid additional gross disturbance of the soil sample and/or collapse.

Furthermore, even small changes in stress resulting in soil deformation can destroy the `memory' of previous stress applications, leading to considerable reduction in stiffness in sands (Clayton et al., 1995; Hight and Leroueil, 2003; Yoshimi et al., 1994). Even with very thin walled push tubes or large diameter rotary samplers (such as the Laval sampler - Konrad et al., 1995; Wride et al., 2000), such effects have been observed to affect the measured soil response.

These issues favoured the development of the freeze sampling technique, where samples are frozen in-situ before being retrieved by coring through the frozen soil mass, and/or retrieving using a crane. By freezing the soil slowly prior to sampling, the pore water is allowed to migrate away from the freezing front thereby preventing the damaging effects of ice-lens formation and frost heave, and the structure, fabric and density of the sand may be preserved. Providing sufficient care is taken in the handling of the frozen samples, particularly in trimming the samples in the laboratory, then very high quality data can be obtained (e.g. Goto, 1993; Singh et al., 1982; Yoshimi et al., 1978, 1994; Hatanaka et al. 1985; Wride et al. 2000a). These studies also highlighted the significance of sampling induced strains, where if the volumetric expansion exceeded 0.5 %, or the shear strain exceeded about 0.1 – 0.2%, the fabric of the soil was damaged, and the strain history was lost, resulting in significant reduction in the cyclic resistance of natural samples when measured in the laboratory (Yoshimi et al., 1994). While freeze sampling has been shown to be excellent for clean sands under sufficient confining pressure, the introduction of even relatively modest amounts of fine-grained material to the soil quickly makes this technique unsuitable, with the finer particles restricting the ability of the pore water to migrate away from the freezing front as it transitions from the fluid to solid phase, allowing frost heave effects to damage the soil structure (Singh et al. 1982; Yoshimi et al. 1994; Wride et al. 2000a).

In very fine grained soils, such as soft to firm ***clays***, very good results can be obtained using conventional thin walled tube samplers, with the disturbance associated with pushing the tube restricted to a very thin layer at the edge of the sample, and results from oedometer and undrained shear testing being very similar to the results obtained from high quality block samples. Here the geometry of the sample tubes has been found by many researchers to be critical to successful high quality sampling (Andresen, 1981; Baligh et al. 1987; Clayton et al. 1998; Hight and Leroueil 2003; Hvorslev, 1949; Lunne and Long, 2005; Tanaka et al. 1996, among others). A large sample diameter and sharp cutting angle were considered most critical to obtaining high quality samples where the behaviour in laboratory testing matched those of samples obtained from block sampling. Conventional "Shelby" sample tubes (i.e. ASTM D1587) were found to result in significant disturbance in many of these comparisons. Commonly cited geometry indices are documented in Appendix A1, along with the recommendations of the Norwegian Geotechnical Institute (NGI) and the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE).

Intermediate soils, notably ***silty-sands*** and ***sandy-silts*** fall somewhere between the challenges posed by undisturbed sampling of clean sand-like soils, and the relatively well characterised successful sampling techniques that have been developed and documented for clay-like soils. Despite the range in techniques available for soil sampling, it is recognised that sampling ***silty sands*** below the water table remains problematic, with results likely to be influenced by disturbance to a greater or lesser extent. A mechanism likely to be responsible for the difficulties in sampling these soils is the large frictional forces which can be generated as soil is captured within the sample tubes/ core liners associated with push-tubes or rotary sampling methods. To address this specific mechanism of disturbance, a new sampling technique referred to as "gel-push" (GP) has been developed by Kiso-Jiban consultants in Japan. While still a new technique, GP samplers have been used on a number of projects in Japan (Lee et al., 2012; Chiaro et al., 2014), Taiwan (Huang, 2006), Bangladesh (Silva et al., 2010), Poland (Jamiolkowski, 2014) and New Zealand (Taylor et al., 2012, Stringer et al., 2015).

2.2.1 Conventional fixed-piston drive sampling

Fixed piston samplers of one sort or another have been operating for a long time in geotechnical engineering practice (Olsson, 1936). Modern fixed piston samplers generally refer to the *Osterberg-*

type hydraulically activated piston samplers (Osterberg, 1952; 1973; Osterberg and Varaksin, 1973; Ishihara et al., 1979), and is described in detail by Clayton et al. (1995) and the US Army Corps of Engineers design manual (EM1110-1-1804, US Army 2001). They operate by pushing a steel sample tube into virgin soil down a borehole. Sufficient adhesion between the sample and the sample tube is required to retrieve a sample to surface without the aid of a core-catcher. Osterberg sampling of sands was reported by Ishihara et al. (1979) in predominantly loose to medium dense sands ($N < 15$, D_R 34 - 79%); however no assessment of sample quality was conducted.

2.2.2 Rotary samplers

Rotary samplers are employed in virtually any type of soil and rock, and much denser/ stiffer materials than are possible with a fixed piston sampler. Different types of rotary samplers exist, largely differentiated based on the position of the inner barrel or sample tube; core barrels with retracted inner barrels, (e.g. conventional double-tube swivel type); core barrels with protruded inner barrels (e.g. Denison triple-tube swivel type); and spring-mounted inner barrels, so that it protrudes in relatively soft ground, but retracts when harder layers are encountered (Clayton et al., 1995). The disadvantage with these methods is that when soil conditions are difficult, both equipment and technique must be chosen with care: Fluid flush; rig stroke; barrel length, diameter and design; and bit-type are all important factors (Clayton et al., 1995). Core-barrels tend to have a larger area ratio (AR) and inside clearance ratio (ICR) than is generally accepted for drive samplers (refer Appendix A1 for definitions). The larger inside clearance can cause problems with interaction of fluid flush and soil, effects of vibration on the core as it is less supported, dilatant materials are allowed to expand due to shearing and stress relief. Ishihara (1985) notes the use of an improved Denison type sampler (triple tube) in Japan to obtain undisturbed samples of sand below the water table. He notes it is claimed to be able to retain samples from loose to very dense and cemented sand, but that there is likely to be disturbance to samples obtained from loose deposits due to rotation of the core bit and jetting of mud during the core operation. Marcuson and Franklin (1979) note that these samplers are most successful in soils with cementation or some material cohesion to bind the particles to each other. However, Ventouras and Coop (2009) showed that disturbance of soil fabric appeared to be more pronounced for rotating core samples than for block samples.

2.2.3 Other Methods

To eliminate the source of disturbance due to tube insertion, attempts have been made to obtain samples by either block sampling or ground freezing. Ishihara (1985) cites Horn (1978); Espana et al., (1978); Marcuson and Franklin (1979); and Mori and Ishihara (1979) as documenting the block sampling of sands. Clough and Sitar (1981) and Cresswell and Powrie (2004) used block sampling for cemented or highly interlocked soils. Bradshaw and Baxter (2007) also present the block sampling of silts from above the water table (refer Figure 1). The method involves isolating a column of soil by excavating the surrounding material and encompassing the soil column by a section of tubing, or square box, and finally cutting the bottom free. Excavation must expose the material, requiring test pits, exploratory shafts or trenches. The disadvantage is the large amount of excavation required, sometimes together with a lowering of the elevation of the ground water table. This procedure is expensive and time consuming, particularly below the water table. Block samples are still subjected to changes in the effective stress due to excavation and ground water lowering.

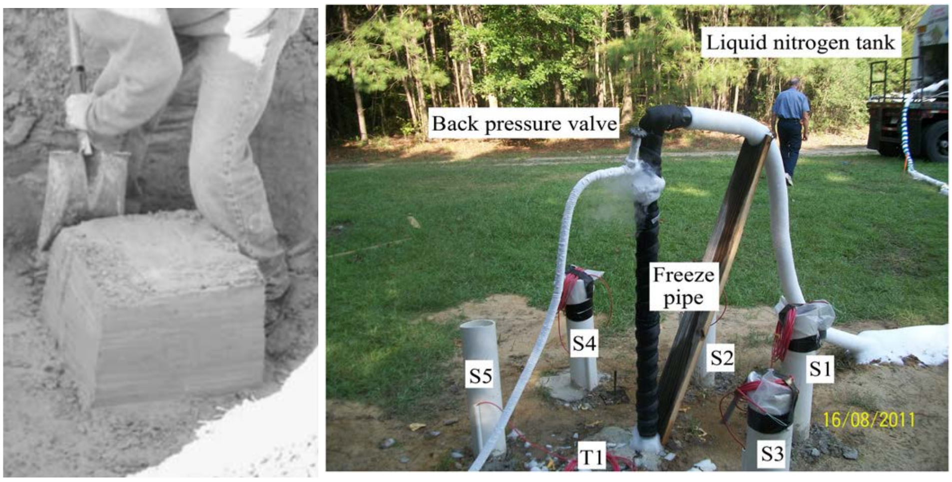


Figure 1: Left: Photograph of block sampling in the field (Bradshaw and Baxter, 2007), Right: Photograph of ground freezing in the field post coring of sample tubes (S1-S5) in South Carolina (Esposito et al., 2014).

The freezing technique has been discussed earlier in this chapter, it is also non-trivial to conduct and very expensive, making it cost prohibitive for most projects. Recently, Esposito et al. (2014) estimated the cost of a ground freezing programme conducted in Charleston, Southern Carolina (one site where five frozen core samples were retrieved from between 1.8 and 3.8m below ground surface) to be US $32,000 including instrumentation, materials, labour, and equipment for installation and operation of the ground freezing system, and to obtain the frozen samples (i.e. drilling). Of which $19,000 was for the liquid nitrogen. The total does not include the engineering oversight, planning or analytical effort. Lower costs can be obtained in colder climates where less liquid nitrogen is required to freeze the ground.

2.2.4 Introducing gel-push sampling

Gel-push (GP) sampling is a recent development in the sampling of saturated cohesionless soils (Tani and Kaneko, 2006). The GP samplers are a variation on existing rotary and drive samplers, with the use of a gel-polymer lubricant during the sampling process. The use of the gel-polymer is primarily to reduce undesired frictional shear resistance between the sample core and the inside surface of the sample tube as it enters the internal liner, with some added benefits of improving the rheological characteristics of the material through increased viscosity, to assist in the development of suction effect to aid in retrieval of a sample from downhole, and with the formation of a film aid in moisture retention and sampling of solids (Sakai, pers. comm., 2011).

GP sampling is currently carried out with one of three types of samplers; GP-S, GP-TR and GP-D, with the key and common feature of the samplers being the delivery of a lubricating polymer gel to the bottom end of the samplers. Figure 221 shows an example (from sampling in Christchurch) of the gel coating the bottom end of the sample using the GP-S sampler. The gel coats the sample, with the aim of significantly reducing the friction between the sample and core barrel. It should be noted that once retrieved from the tool, the sample shown in Figure 221 slid easily in and out of the tube with only minor slope angles from the horizontal. While the gel-push concept overcomes some of the deficiencies of conventional techniques, it has to be acknowledged that the method is still under intense scrutiny and improvement through continuing field-based studies. To date, sampling has been carried out with both the GP-S and GP-TR samplers in Christchurch and the details of the sampling processes associated with these samplers is covered in the following sections.

Figure 2: Lubricating polymer gel coating soil sample

2.3 Samplers trialled in this study

Three sampler configurations have been trialled for this research project, two recently developed samplers based on the gel-push sampling concept, GP-S and GP-TR, and an existing thin walled tube piston sampler for fine grained soils, referred to as the *Dames & Moore* piston sampler.

2.3.1 Gel-Push piston sampler (GP-S)

The GP-S sampler follows the concepts of fixed-piston sampling, and shares many of the features of the Osterberg sampler. The GP-S sampler comprises three barrels, a fixed piston and a two travelling pistons, as shown in Figure 331. The inner core-liner barrel consists of a PVC pipe with an inner diameter (ID) of approximately 70mm, and a total length of 990mm (L/D ratio: 14.1). A particular feature of the core-liner barrels are a series of holes located near the top of the sample liner tube, which allow the outflow of polymer gel from the inner barrel during the sampling process.

Upon initial assembly, the middle barrel of the sampler is connected to the upper travelling piston, which is at the uppermost extent of its travel. The middle barrel is filled with the polymer gel, typically mixed to a concentration of between 1 and 3 percent by volume, and the core liner barrel is inserted while rotating to ensure that both the inner and outer surfaces of the core liner barrel are well coated with the polymer gel. When fully inserted within the sampler, the top of the core liner barrel rests on the bottom face of the lower travelling piston. To prevent the soil samples from dropping out of the tube after sampling, a steel core catcher is fitted to the bottom end of the core liner barrel, using a collar to ensure correct alignment of the two pieces.

The fixed piston of the sampler is fitted to the end of the fixed piston shaft, and features an internal mechanism to enable the core-catcher activation process. Finally, a cutting shoe is attached to the bottom end of the middle barrel.

The geometry of the GP-S in relation to the descriptions noted in Appendix A1 is presented in Table 2 alongside the other samplers discussed in this report, and several leading samplers mentioned in the literature.

Due to the relatively thin wall thicknesses of some components within the tool, it is important that the hydraulic pressure used to drive the tool is limited to 7MPa. The authors recommend that the tool only be used to obtain soft soil deposits where the CPT cone resistance is approximately 5MPa or less.

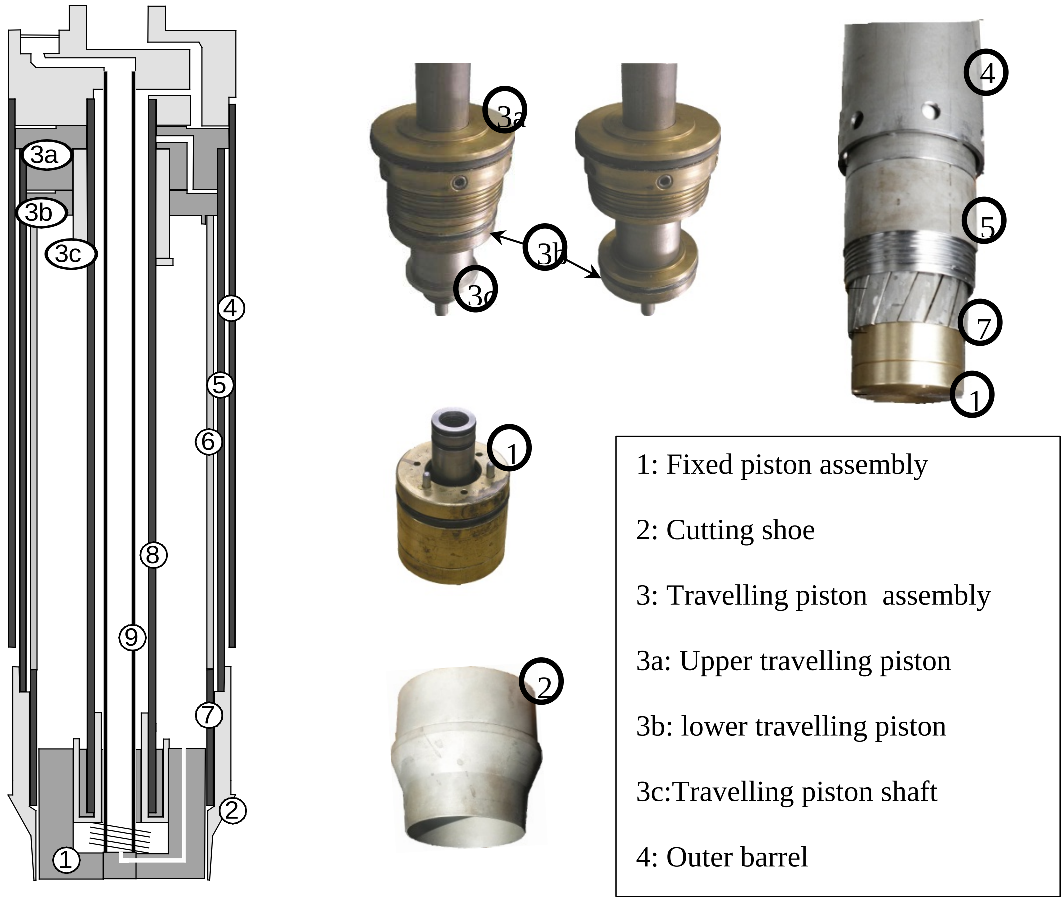


Figure 3: GP-S sampler schematic and key components

2.3.2 Gel-push triple tube sampler (GP-TR)

A second gel-push sampler, known as GP-TR was designed based around the concept of a Denison-type triple tube sampler with a spring mounted cutting shoe, that varies the projection of the cutting shoe into the soil depending on the soil stiffness or density. The specification calls for a minimum projection of 40mm into the soil ahead of the rotating bit and effects of drill flush. For rock or very hard soils it is possible for the cutting shoe to fully retract allowing for disturbance induced by the rotary drilling, and sampling in these materials is not recommended. It features a rotating reaming shoe and a no-rotation inner core liner barrel. The main components of this sampler are shown in Figure 4.

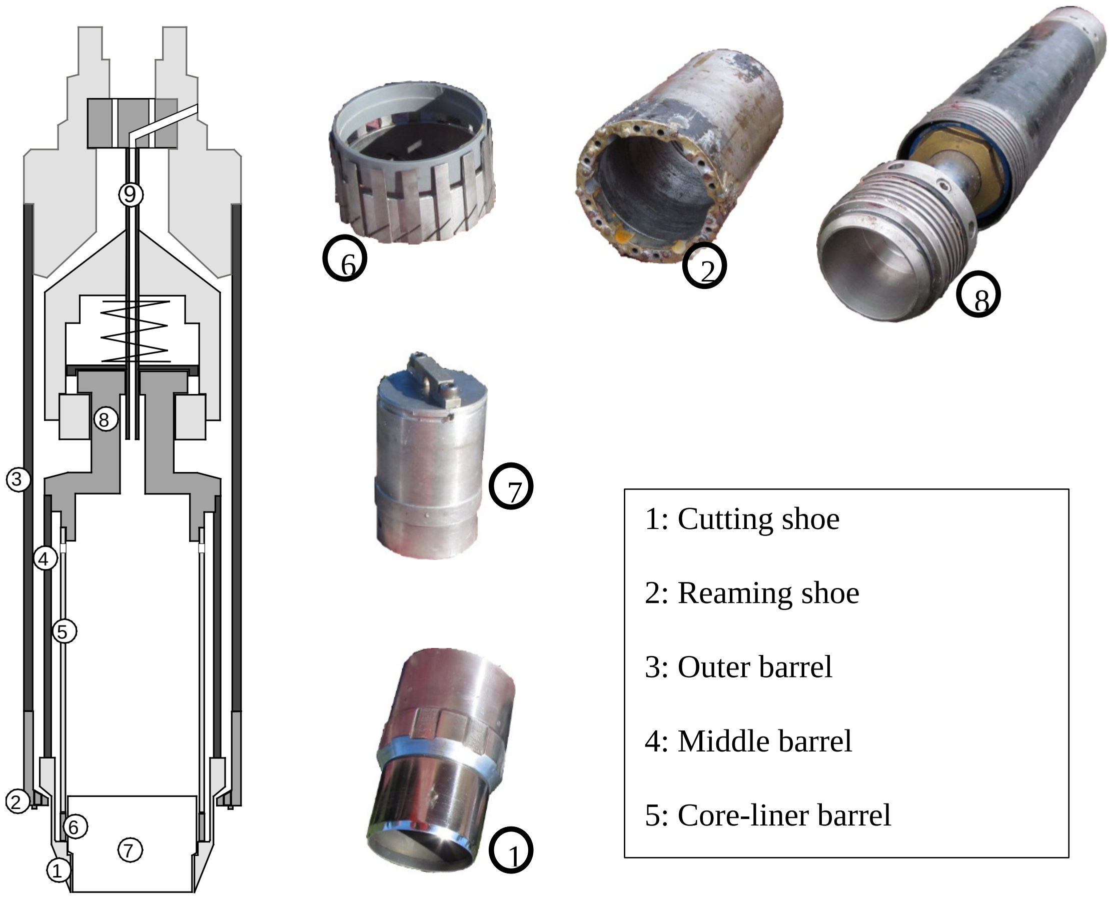


Figure 4: GP-TR schematic and key components

Similar to the GP-S sampler, the inner core liner barrel is housed within a middle barrel which is filled with polymer gel prior to insertion of the core-liner barrel, which in this case is 1m in length and 72mm inner diameter. A short extension piece is placed between the cutting shoe and the core-liner barrel with a number of fine slots to enable the passage of gel to the sample. A "floating" piston is placed within the cutting shoe during assembly, such that it sits at the bottom of the sampler when the sampler is lowered into the borehole.

2.3.3 The Dames and Moore piston sampler

The Dames and Moore sampler is a hydraulically activated fixed-piston device. When hydraulic pressure is applied to the sampler, the core-liner barrel is pushed directly into the soil. The core-liner barrels are 50cm in length, with a 63mm inner diameter and approximately 1.5 mm wall thickness. The Dames and Moore sampler attempts to minimise the problems associated with side wall friction by reducing the length of the core-liner barrel, and in some cases using brass tubes which exhibit a lower interface friction angle.

Figure 5: Dames and Moore Sampler

Table 2: Comparison of key sampler dimensions

Sampler	Sampler Length (mm) L	Thickness (mm) t	Inside diameter (mm) D_c	Outside Diameter (mm) D_e	Inside Clearance ICR	B/t Ratio B/t	L/D Ratio L/D	Area Ratio, (%) AR	Outside cutting edge angle (º) OCA	Sampler type
JPN[1,3]	1000	1.5	75	76.5	0%	52	13.1	7.5	6	Piston
Laval[1,4,]	660	4	208	218	0%	54	3.1	10	5	Rotary
Shelby[2]	910	1.65	72	76.2	<1%	46.2	11.9	14.2	21	Push
GP-S[6]	1000	11.8	71.2	95	0.28%	8.1	10.5	78	~14.5	Piston
GP-TR[6]	1000	14.8	72.1	101.6	1.50%	6.9	9.8	105	~20.9	Rotary
Dames & Moore[5]	450	1.5	61.2	63.5	0%	42.3	7.1	7.6	~30	Piston
NGI 54[1.,3.]	768	13	54	65	0.60%	6.2	11.8	44	5	Piston
Geonor 76[1]	585	2	76	80	0%	40	7.3	11	15	Piston
NGI 95[3]	1000	2.6	95	101.6	0.8-1.3%	38.5	9.8	11	16	Piston
Sources: 1. Tanaka et al. 1996; 2. ASTM D1587; 3. Lunne et al. 2006; 4. Konrad et al. 1995; 5. Bray and Sancio 2006; 6. Kiso Jiban Consultants GP-S and GP-TR documentation.										

2.4 Methodology

2.4.1 GP-S Sampler

When taking a sample, the GP-S sampler is lowered into the borehole to the depth of last rotary drilling, pushing the sampler past any slough which may have fallen into the hole. When the sampler has reached the required sampling depth, the drill pipe is locked in place at the surface and the drill pipes are filled with water before being connected to a water pump. Before starting the pump, a bypass valve is opened so that flow is initially returned to the water reservoir. The bypass valve is gradually closed, so that pressure builds on the upper travelling piston. This pressure results in the middle barrel advancing the cutting shoe into the soil, while leaving the inner core barrel unstressed. The initial phase of GP-S sampling is depicted in Figure 5. It should be noted that the parts of the sampler which are moving have been coloured blue in Figure 5.

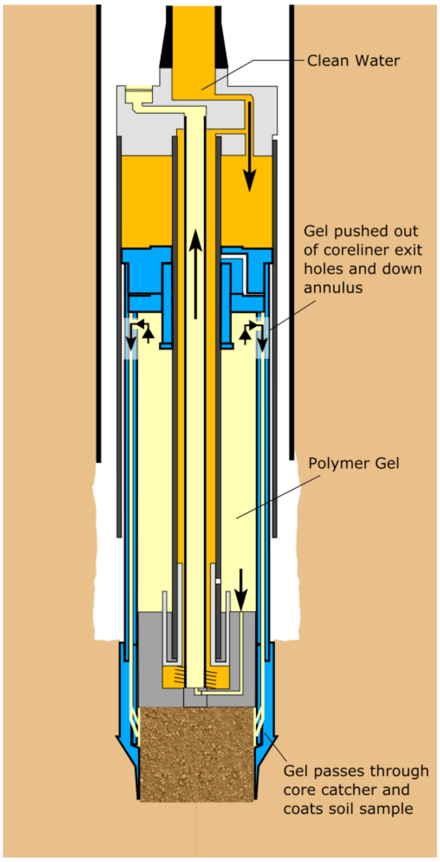


Figure 5: GP-S during drive phase

While the middle and inner barrels advance, the fixed piston remains in the same location, hence the volume within the core barrel occupied by the gel reduces by forcing the gel out of the holes at the top of the core barrel and down the annulus between the inner core barrel and the middle barrel, also shown in Figure 5. When the gel reaches the core catcher, it passes through the gaps between the catcher "fins" (see photo of core catcher in Figure 7) and coats the surface of the soil sample as it enters the core barrel. It should be noted that an O-ring seal on the outside of the fixed piston "wipes" the inside of the core barrel as it advances past the fixed piston, meaning that gel cannot pass the fixed piston in either direction. During sampling, only a small fraction of the total gel placed within the tool is intended to coat the sample. The excess gel must therefore be vented from the tool to prevent large pressures being exerted onto the sample. This takes place through the fixed piston (as shown in Figure 5), which allows gel to pass through its upper face and enter a small diameter conductor pipe within the fixed piston shaft and exit into the borehole through the top of the tool.

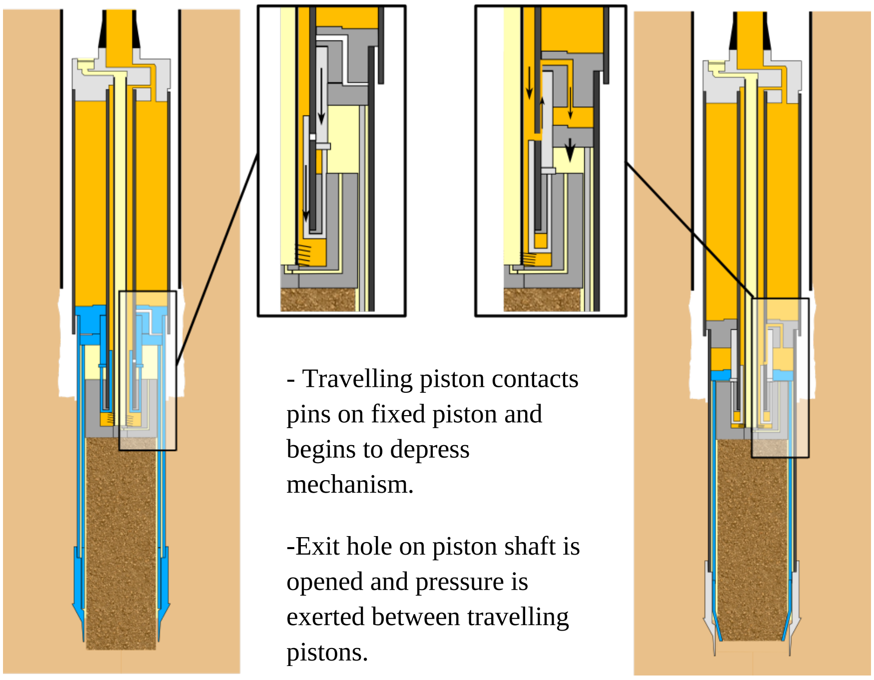


Figure 6: GP-S core catcher activation

Once the tool has advanced 1m**,** the base of the travelling piston assembly comes into contact with the spring-loaded pins on the fixed piston, as depicted in Figure 6. When these pins are depressed, the fixed piston sleeve is moved downwards, opening an exit port on the fixed piston shaft which allows fluid to reach the area between the upper and lower travelling pistons. Fluid pressure now acts at the interface between the two travelling pistons and causes the lower travelling piston to apply a downwards acting load on the core-liner barrel, which is transferred to the core catcher. This load causes the core catcher to move downwards, while the chamfered inner surface of the cutting shoe forces the core catcher blades inwards, securing the sample within the barrel as demonstrated in Figure 7.

At the end of sampling, the fixed piston remains entirely within the core liner barrel, and in the case of 100% recovery, a 92 cm long sample will be obtained.

Figure 7: Activated core catcher

2.4.2 GP-TR sampling

Figure 8 indicates some of the key features of the GP-TR sampling process. As the sampler approaches the bottom of the borehole, rotation and mud circulation begins, with drilling mud diverted around the middle barrel to a number of jets within the reaming shoe. Once the sampler cutting shoe makes contact with the bottom of the bore hole, the cutting shoe begins to penetrate the soil and soil enters the sampler. The cutting shoe and middle barrel are connected to a spring loaded rotational bearing, meaning that in soft soils, the cutting shoe will be fully extended, while in stiffer soils, the cutting shoe retreats, bringing the reaming shoe closer to the sampled soil face.

During sampling (right hand diagram in Figure 8), the drill pipes are continually rotated at moderate speed (40 - 80 RPM), which is transferred to the outer barrel and reaming shoe, while the rotational bearing isolates the middle and core liner barrels from rotation. Centralisers are fitted on the drill pipe to stabilise the sampler and reduce vibrations.

The rotational bearing is spring loaded in the axial direction. This is intended to provide a more constant loading of the cutting shoe during sampling. A direct consequence is that the extension of the cutting shoe relative to the reaming shoe is not constant. In soft soils, the cutting shoe will penetrate relatively easily and will therefore be close to maximum extension. However, in stiff dense soils, the penetration will be greatly reduced. It should be noted that during sampling, the cutting action of the drilling mud jets are expected to be responsible for most if not all of the hole extension.

The floating piston remains stationary during sampling, resting atop the soil which has entered the sampler as it advances. The travel of the floating piston relative to the core liner barrel displaces the polymer gel, in a similar way to the GP-S sampler, forcing it down the annulus between the middle barrel and core-liner barrel. The gel exits through the slots in the gel delivery spacer and coats the outside of the sample. Again, excess polymer gel is vented, this time through the conductor pipe in the bearing assembly, exiting through the top of the sampler.

Sampling is ended once the sampler has advanced 1.05m, after which the sampler is gently pulled from the borehole. It should be noted that this sampler does not incorporate a core catcher, hence it is necessary to rely on suction within the sample to prevent dropout.

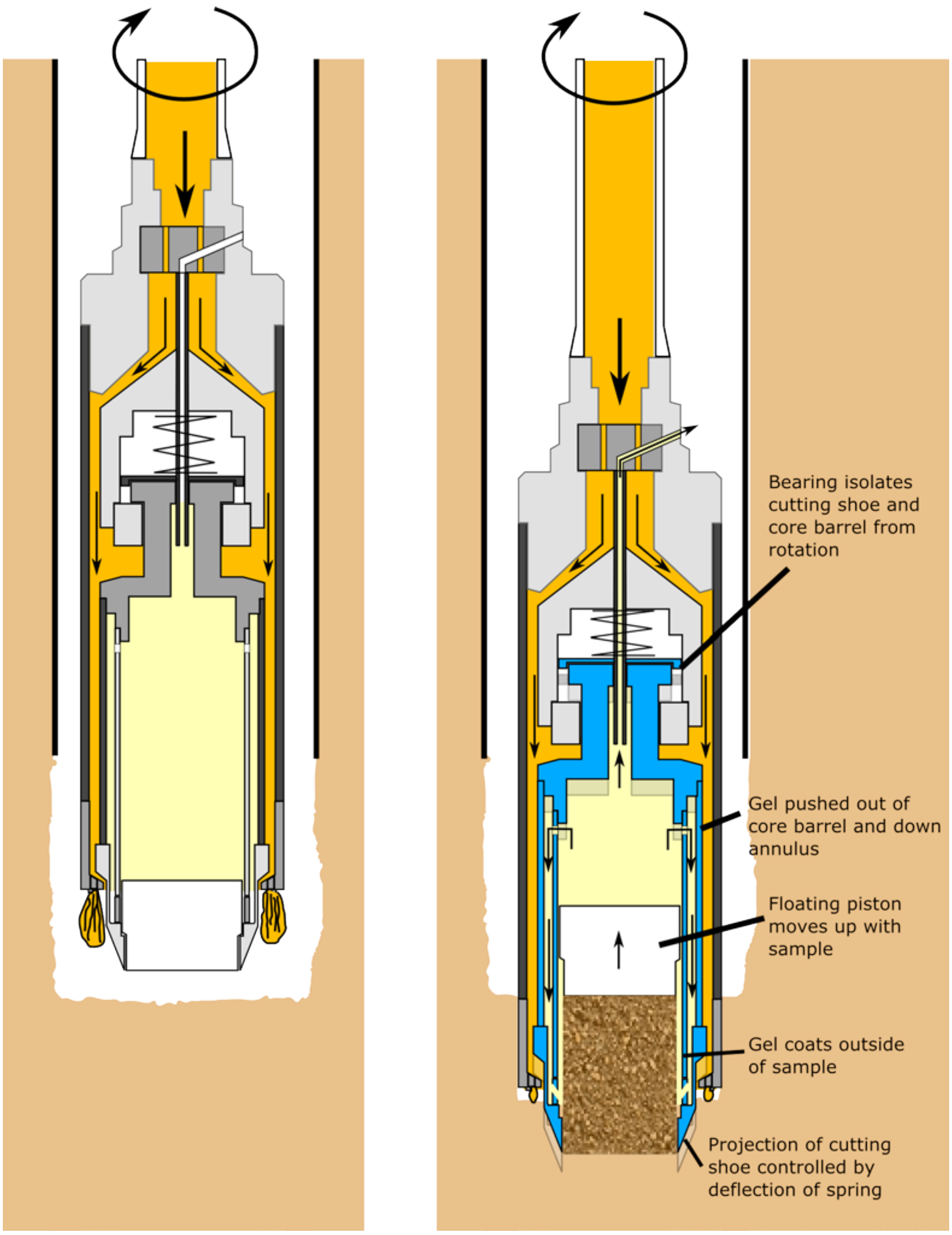


Figure 8: GP-TR during sampling

2.4.3 Dames and Moore sampling

During sampling, the fixed piston (visible at the bottom of the sampler in Figure 5) remains in the same position, while a travelling piston is driven downwards by the application of hydraulic pressure from the drilling rig. The brass core barrel is attached to the travelling piston and is driven into the soil by the advance of the travelling piston. The sampler is advanced as quickly as possible and sampling ends when the travelling piston reaches the holes visible near the bottom of the outer barrel, which release the pressure driving the piston and core barrel. After sampling, it is necessary to allow a significant time period to elapse before attempting to retrieve the sampler from the borehole to reduce the chances of sample “dropout”.

2.5 Preserving sample quality after sampling

2.5.1 Onsite

Once retrieved from the borehole, the sampler (either of the GP-S or GP-TR samplers) is carefully laid out to lie horizontally, and the cutting shoe and middle barrel unscrewed from the sampler. At this point, the fixed piston remains inside the sample barrel. A polystyrene plug is used to “fix” the end of the sample, while sliding the outer barrel off the fixed piston. Tubes are carefully raised back into their vertical orientation and allowed to drain on-site (overnight) before plastic end caps are used to seal both ends of the core-liner barrel.

2.5.2 Transportation

Gel-push samples are typically allowed to drain onsite before transportation back to the laboratory, with drainage enabling some level of effective stress to be re-established. Samples are transported vertically within a car, with a folded duvet and layers of clothes placed beneath their container in an effort to isolate the samples from the road vibrations. Bubble wrap is typically used to reduce lateral vibrations on the samples.

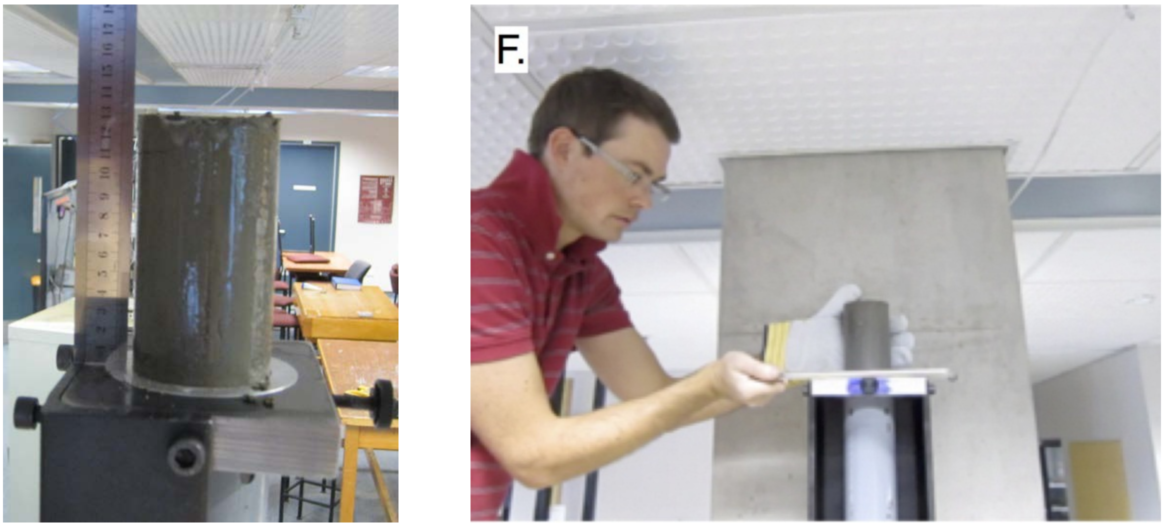
2.5.3 Laboratory handling

Samples are stored in a vertical orientation in the laboratory and remain within their core liner barrels until the sample is due to be tested.

At this stage, the soil sample is extruded from the core liner barrel using a vertically mounted piston (Figure 9), which gradually pushes the soil out of the tube, ensuring that the soil is pushed from the tube in the same direction as it entered (i.e. soil is pushed out of the top of the sample tube). The top 10cm and bottom 5cm of each soil are discarded, since this soil is thought to have experienced some disturbance, either as a result of the drilling prior to sampling, or the processes involved at the end of sampling. As the soil is pushed out of the tube, it is cut into a number of samples using a wire saw (Figure 10). Soil samples are examined for obvious deficiencies (Figure 11), which may arise out of the sampling process, or for inclusions which may be problematic for the testing process (such as the presence of large twigs, or cobbles). Extrusion of a gel-push sample must take place in one operation (i.e. samples are not incrementally extruded and tested as required). Samples are preserved by wrapping them in cellophane (cling-film) and storing them in a humid environment.

(a) **(b)**

Figure 9: Extruding a soil sample (Taylor, 2015)



(a) **(b)**

Figure 10: Cutting soil samples during extrusion (Taylor, 2015)

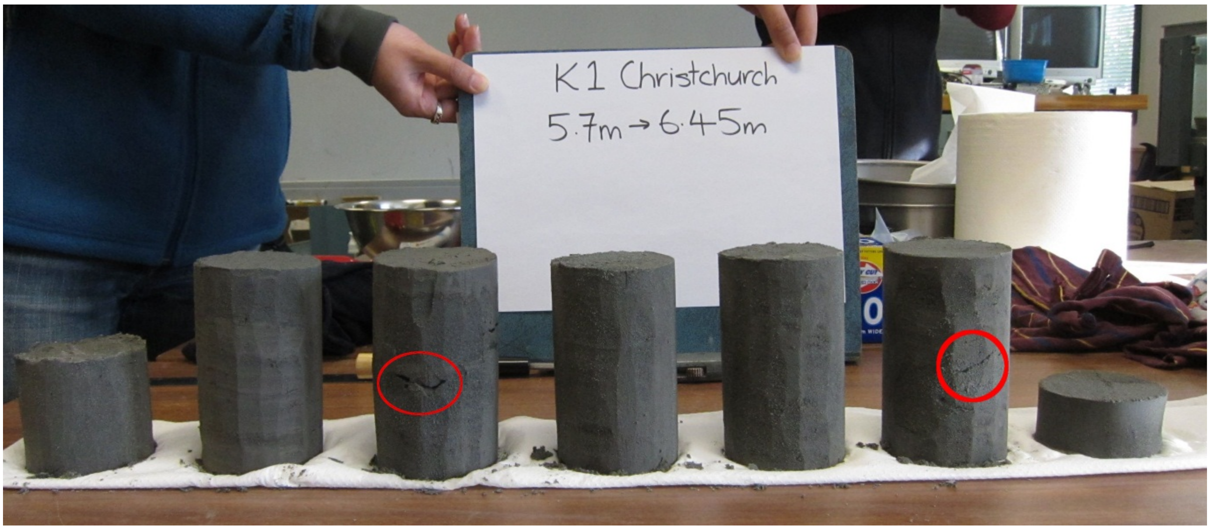
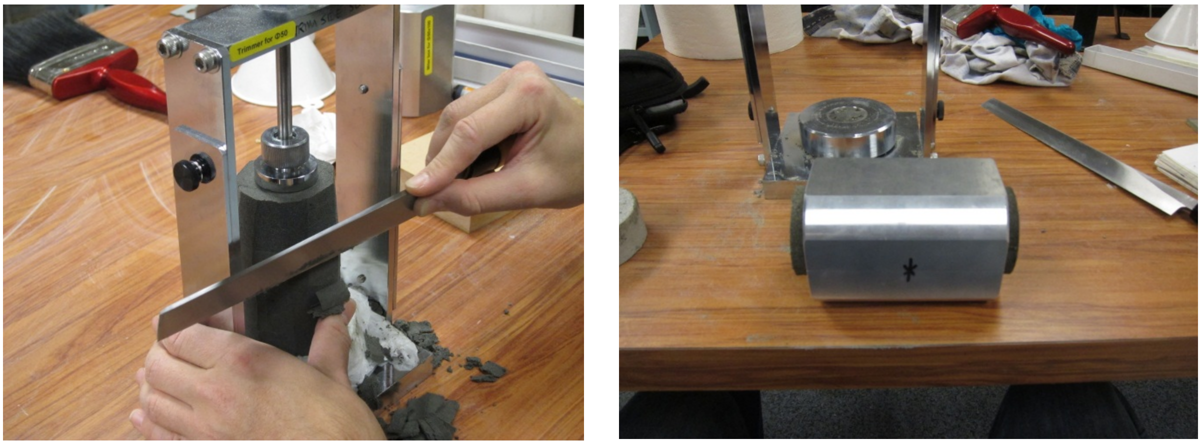


Figure 11: Examining extruded samples for defects (Taylor, 2015)

The use of polymer gel in the sampling process can lead to impregnation of an outer layer of soil. This zone of soil is removed by radially trimming the sample with a sharpened straight-edged knife (Figure 12). Immediately after radial trimming, the samples are placed within a mitre box and shortened so that the length is twice the sample diameter.



(a) **(b)**

Figure 12: Trimming the sample (Taylor, 2015)

In the case of clean sands, the polymer gel can extend significantly into the soil samples, such that some gel remains within the sample after trimming has been completed. However, it is thought that most of the remaining gel is flushed out of the samples during saturation.

The trimmed sample is carefully encased within a thin latex membrane before being mounted on the triaxial device and sealed.

2.5.4 Dames and Moore sample handling

In order to reduce the loads acting on the soil sample during extrusion, the tubes are cut using a pipe cutter at pre-determined locations (often decided using the results from an exploratory CPT) and using confining rings to prevent ovaling of the samples. A wire saw is used to cut through the soil at the location of the cut in the tubing.

The Dames and Moore samples are typically tested without further trimming of the sample.

3. Sample quality evaluation

It is necessary to consider the likely degree of disturbance of soil samples obtained from the field to understand the degree to which the test results conducted on the samples are representative of in-situ conditions. While there is no direct measurement of sample disturbance, a number of methods have been used by other researchers to attempt to quantify sampling disturbance. These include assessing the change in sample density, the degree to which effective stresses have been maintained within the sample and the change in the soil's small strain stiffness.

The choice of method is dictated in part by the characteristics of the sampled soil. In clays and some silts, the degree to which effective stresses are maintained can be a reasonable parameter to evaluate disturbance. This is assessed by either measuring the changes in void ratio as the sample is brought back to in-situ stress levels during testing, or application of a suction probe to the sample. This method is appropriate for clays due to the high air-entry value of the soil, meaning that changes in effective stresses in the sample are likely to have been induced by shearing rather than the entry of air/fluid into the sample.

In soils where air entry values are lower, such as silty soils, alternative methods of assessing sample disturbance must be used. Direct comparisons of sample density with the field are desirable, given its strong links with soil behaviour. However the uncertainty in the measurement of the density using available field techniques is unlikely to be acceptable for the purposes of determining the level of disturbance.

A comparison of shear wave velocity has therefore been chosen as the metric for assessing disturbance. Shear wave velocity is related to the small strain stiffness of a material via elastic wave theory, and may be represented simply via Equation 1. The small strain stiffness is known to degrade significantly with shear strain and therefore disturbance.

$$V_s = \sqrt{\frac{G_0}{\rho}} \tag{1}$$

Shear wave velocity is known to be affected by the level of confining stress acting on the sample and as such it is common to compare the values of shear wave velocity corrected to atmospheric pressure (denoted $V_{s,1}$). The correction commonly used is derived from the following relationship (after Roesler, 1979; Lee and Stokoe, 1986; Jamiolkowski et al. 2014):

$$V_{s,1} = V_s \left[\left(\frac{P_a}{\sigma'_a}\right)^{n_a} \cdot \left(\frac{P_a}{\sigma'_b}\right)^{n_b} \cdot \left(\frac{P_a}{\sigma'_c}\right)^{n_c} \right] \tag{2}$$

The out-of-plane principle stress is ignored for vertically oriented shear waves, simplifying the relationship to:

$$V_{s,1} = V_s \left[\left(\frac{P_a}{\sigma'_{v0}}\right)^{n} \cdot \left(\frac{P_a}{\sigma'_{h0}}\right)^{n} \right] \tag{3}$$

The horizontal stresses may be approximated from the vertical, by adopting a horizontal earth pressure coefficient, K_0 for the material:

$$V_{s,1} = V_s \left[\left(\frac{P_a}{\sigma'_{v0}}\right) \cdot \left(\frac{P_a}{K_0 \sigma'_{v0}}\right) \right]^{2n} = V_s \cdot K_0^n \cdot \left(\frac{P_a}{\sigma'_{c0}}\right)^{2n} \tag{4}$$

For field conditions comprising normally consolidated sandy soils, K_0 is typically assumed to be equal to about 0.5, n = empirical stress exponent = 0.125, and the following is typically adopted using field data (Robertson et al., 1992; Andrus and Stokoe, 2000):

$$V_{s,1} = V_s \left(\frac{P_a}{\sigma'_{v0}}\right)^{0.25} \tag{5}$$

These corrections are important when comparing measurements in the laboratory with those measured in-situ. In the laboratory tests described in this report, the soil samples are consolidated isotropically (i.e. K_0 = 1) to a confining stress, σ'_{c0}, value slightly larger than the vertical effective stress estimated in the field. Additionally, in the field, the vertical and lateral effective stresses are generally quite different. For the soils encountered in Christchurch, it is assumed that the soils are normally consolidated and that the horizontal effective stresses can be estimated according to Equation 5.

While experience in New Zealand is still limited, the GP-S and GP-TR samplers have been used at a number of sites within Christchurch (preliminary results are presented later). In particular, silty soils were retrieved using the GP-S type sampler at Gainsborough Reserve. The CPT data suggests alternating layers of silty sand ($I_c \approx 2$) and silty clay/organic material ($I_c \approx 3$), as shown in Figure 13.

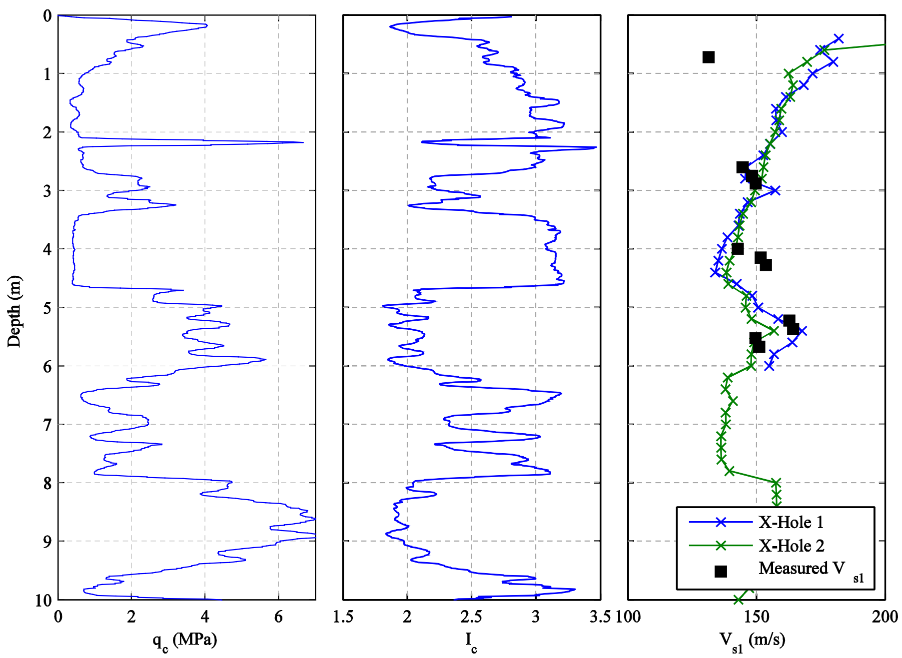


Figure 13: Soil profile at Gainsborough Reserve and shear wave velocity comparison

A number of soil samples were retrieved from this site, and tested in the laboratory. Comparisons of normalised shear wave velocities from the laboratory and in-situ measurements (cross-hole technique) are shown in Figure 13. The results indicate that with the exception of the shallowest specimen, the gel-push (GP-S) sampling has been relatively successful in preserving the soil's in-situ shear stiffness in these soils. The precise reasons why the shear wave velocity of the shallowest specimen was lower than expected remain unknown. However, some twigs were found to run through the specimen, as well as a large number of small rootlets and holes within the specimen. It is possible that these features made that particular sample particularly prone to disturbance during sampling.

The GP-TR sampler has been trialled at two sites in the Red Zone[1], where clean sands were encountered. A similar assessment of sample disturbance has been carried out at this site, and is shown in Figure 14. However, it is apparent that the shear wave velocity of the samples has not been well preserved. There are a number of potential causes for this result, including some procedural details during sampling, transportation and the extrusion process.

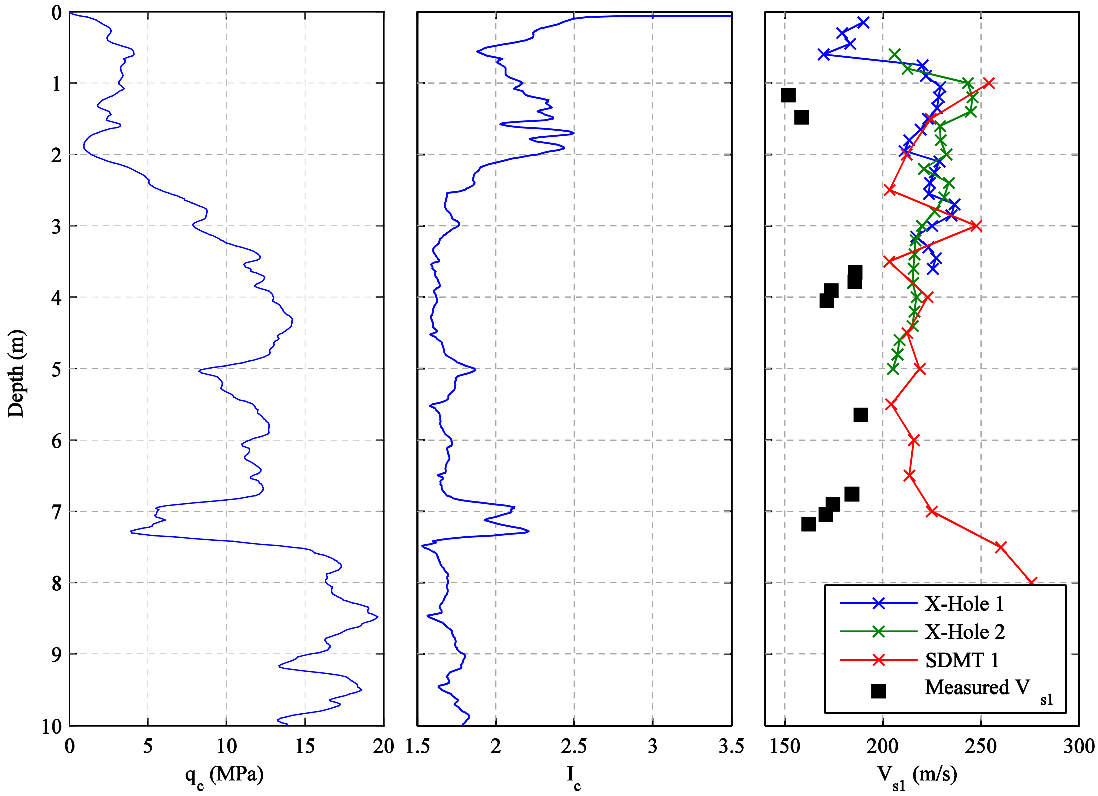


Figure 14: Soil profile from Ardrossan St with comparison of in-situ and laboratory shear wave velocities

While sampling with the GP-TR has not yet been successful in obtaining undisturbed samples, the tool has been used successfully by a number of researchers, notably at the Zelezny Most tailings dam in Poland (Jamiolkowski, 2014), where a similar comparison of laboratory and in-situ shear wave velocities yielded good results. It is expected that with additional experience, high-quality samples will be possible using the GP-TR sampler.

[1] Abandoned suburbs near the Avon River in eastern Christchurch due to extensive and severe liquefaction damage to land, dwellings and infrastructure following the 2010-2011 Canterbury Earthquake Sequence.

4. Testing procedures for undisturbed samples

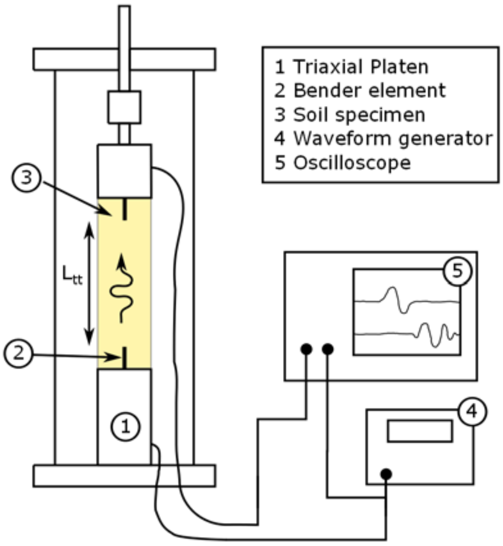
4.1 Saturation and consolidation

Soil samples must be saturated and brought to equilibrium under elevated pressure prior to testing, so that the conditions during testing are reasonably close to those expected in the field.

During saturation, the sample is either percolated with carbon dioxide gas, or subject to a high vacuum, after which water typically flows through the sample under a small head difference. Saturation is completed by increasing the water pressure in the sample, while maintaining the effective stress; a process known as back-pressure saturation. Saturation is deemed adequate once the “B-Value” exceeds a minimum of 0.97. At this point, the saturation ratio (ratio of fluid volume to total void volume) of the sample is typically in excess of 99.5%.

During consolidation of the sample the mean effective stress is raised in a number of discrete steps. When testing undisturbed samples at the University of Canterbury, the samples are typically isotropically consolidated a mean effective stress equal to 1.1 times the estimated in-situ effective stresses.

4.2 Bender element testing



(a) General layout

(b) Bottom platen and bender element

Figure 15: Elements of a bender element system

Bender elements are widely used within triaxial test equipment to obtain measurements of the shear wave velocity of soil samples at specific times during a test sequence. Most commonly, bender element tests are carried out following consolidation of the sample to obtain the small strain stiffness of the soil sample, following Equation (1).

In a typical bender element set-up, wafers of piezo material are embedded within the end platens of the triaxial cell, with one bender element being designated the “source,” and the other the “receiver.” The apparatus used in a bender element test is shown in Figure 15, where the source bender element is located at the bottom of the triaxial device. The source bender element is connected to a waveform generator. During a bender element test, a single sinusoidal voltage pulse is sent by the waveform generator. The change in voltage applied to the source bender element causes it to deflect sideways. The deflection of the bender element creates a vertically propagating shear wave in the soil sample. When the shear wave reaches the receiving bender element, the deflection of the piezo wafer causes a small voltage to be generated. The outputs from the waveform generator and the receiver bender element are connected to an oscilloscope. Following completion of the test sequence, these signals are analysed to estimate the time taken for the shear wave to travel the length of the soil sample between the tips of the two bender elements (shown as L_{tt} in Figure 15), and therefore estimate the shear wave velocity of the sample.

There remains some debate in the literature on the correct interpretation of the first arrival of the shear wave (e.g. Arroyo et al., 2002; Lee and Santamarina, 2005). Typical results from one of the bender element tests are shown in Figure 16. In the figure, 4 points are marked on the received signal which is often used to determine the travel time of the shear wave. For the points marked RA, RB, RC, the travel time is estimated as the time difference from the first increase in the input signal (i.e. point SA), while for point RD (first major peak), the time difference is taken from the maximum in the input signal (i.e. point SB). A number of reasons have been proposed to explain the existence of the small peak observed in the received signal before the deflection in the expected direction (i.e. point RB), including near-field effects around the bender elements, the presence of P-waves and differences between the signal being sent by the waveform generator and the wave being applied to the soil by the source bender element (Lee and Santamarina, 2005; Arulnathan et al., 1998). At the University of Canterbury, travel times are estimated as the difference between points SA and RC.

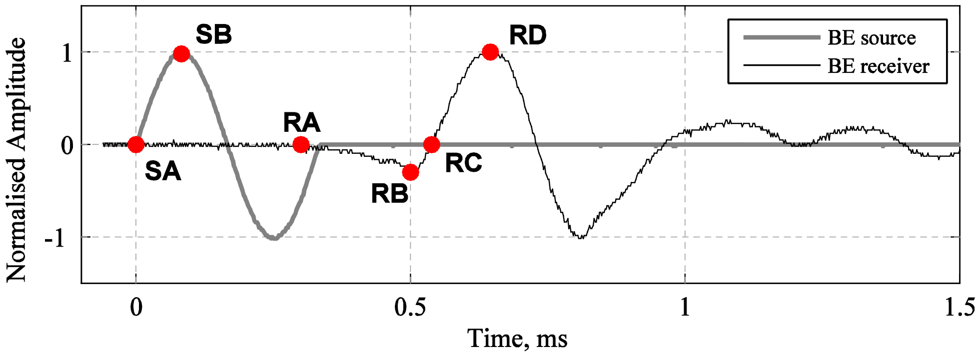


Figure 16: Interpretation of travel time

4.3 Cyclic triaxial testing

Time-varying axial loads are applied to the soil sample by a pneumatically driven loading ram. By applying changes to the axial stress in this manner, a time varying shear stress is applied to the sample, with the maximum shear stress acting on a plane inclined at 45° to the vertical.

The amplitude of loading and the frequency are pre-set by the researcher, and during the experiment, computer software controls the loading via a servo-valve. Cyclic loading continues until halted by the user. Tests aimed at understanding the liquefaction resistance of soils are typically conducted in an undrained condition, with loading cycles applied at 0.1Hz. Tests are halted after a minimum peak-to-peak (i.e. "double amplitude") axial strain of 5% has been observed.

By carrying out a sequence of tests in which similar samples are tested with different loading amplitudes, relationships can be determined to link the number of cycles required to reach a specified strain level to the specific loading amplitude. Examples of these relationships will be shown in later sections.

4.4 Measurement of volumetric strain after liquefaction

Following the termination of cyclic loading, the sample is allowed to consolidate back to its original effective confining stress, while monitoring the volumetric strains which occur.

4.5 Particle size distributions

The particle size distribution (PSD) can play an important role on the behaviour of a soil sample and has therefore been determined for each soil sample tested in the laboratory.

Conventionally, PSDs are obtained using sieves for the coarser fractions of a soil, and by hydrometer for the fines fraction. However, the use of a hydrometer is time consuming, and in the studies described later in this report, particle size distributions were obtained by laser diffraction using an LA-50 Particle Size Analyser manufactured by Horiba.

Small samples of a soil sample (<2g) are dispersed in a solution of sodium-hexametaphosphate (2% concentration by mass) before being mixed in a water reservoir in the machine. The mixture of water and soil solution is continually fed through a glass plate, onto which a laser is directed. When the laser strikes the soil particles, a diffraction pattern is created (e.g. Figure 17) which is associated with the size of the particle. The diffraction pattern is measured by the device, and compared against diffraction patterns which would be expected for different particle size distributions using the Mie theory of light scattering.

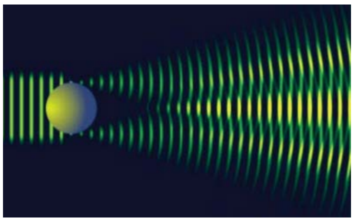


Figure 17: Diffraction pattern around single spherical particle (Horiba, 2014)

The technique presents a number of benefits which includes being able to measure the complete particle size distribution (in conventional analyses, the PSD curve is measured at a number of discrete points) very rapidly. However, it must be noted that there remain a number of issues which must be considered when using these results.

The measurement of particle size is increasingly affected by the material's refractive index as the particles become smaller (but is relatively insensitive in the silt/sand fractions). In materials where the refractive index is either unknown, or soil grains do not have a single refractive index, then the distributions can become skewed. Additionally, the measurement theory assumes that all particles are spherical. This is thought to be the source of many of the differences in the particle size distributions obtained using laser diffraction analysis and conventional sieves/hydrometers. While the distributions obtained by one method (i.e. laser diffraction or conventional sieve analysis) are typically consistent with each other, great care must be taken if the results are to be directly compared with a different measurement technique. These issues are discussed further in Abbireddy et al. (2009). As the database of soils tested in Christchurch becomes larger, it is anticipated that relationships linking PSDs obtained by laser diffraction and conventional sieve analyses will be developed.

5. Advanced Sampling in Christchurch

5.1 Projects conducted

This report presents two research applications of advanced soil sampling conducted by the University of Canterbury since the 2010-2011 Canterbury Earthquake Sequence:

1. GP sampler trials in the Christchurch CBD
2. The Silty Soils Project

Both projects utilise GP sampling. Following the initial trials in the CBD, the Silty Soils Project more extensively utilised both GP-S and GP-TR type samplers, as well as the Dames & Moore thin walled tube piston sampler in sites around Christchurch. Additional application of the GP samplers have been conducted for consulting and research projects with external partners in micaceous silty sands in Queenstown, pumiceous sands in the North Island, and very soft/ weak silts near Belfast, north of Christchurch.

5.2 Field Trials in the Christchurch CBD

Taylor et al. (2012) presented the first field trials of the GP samplers in New Zealand, which were conducted in the Christchurch Central Business District (CBD) in July-August 2011. These trials provided GP samples that were tested in the triaxial apparatus, both for cyclic undrained strength (CTX) and monotonic drained strength (CID), and the sample quality was evaluated using available methods. Detailed comparisons were also made between reconstituted specimens and GP samples to evaluate the influence of natural soil features on the cyclic response in particular. This work has been documented in the Ph.D. thesis of Taylor (2015), and is briefly summarised herein.

5.2.1 Sampling sites

Two sites were selected for the field trials in the CBD. The investigation immediately followed a CPT test programme conducted in conjunction with the University of California Berkeley, which included intensive testing across zones in the CBD where liquefaction-related damage to building foundations were observed (Cubrinovski et al., 2011; Bray et al. 2014). The two sites selected were adjacent buildings affected by extensive and severe liquefaction:

1. Kilmore Street, between Colombo and Manchester Streets. Sampling borehole, K1.
2. Madras-Armagh Street intersection. Sampling borehole, MA1.

Figure 19 shows the locations of these sites, along with the observed liquefaction in the CBD following the 22 February 2011 earthquake event.

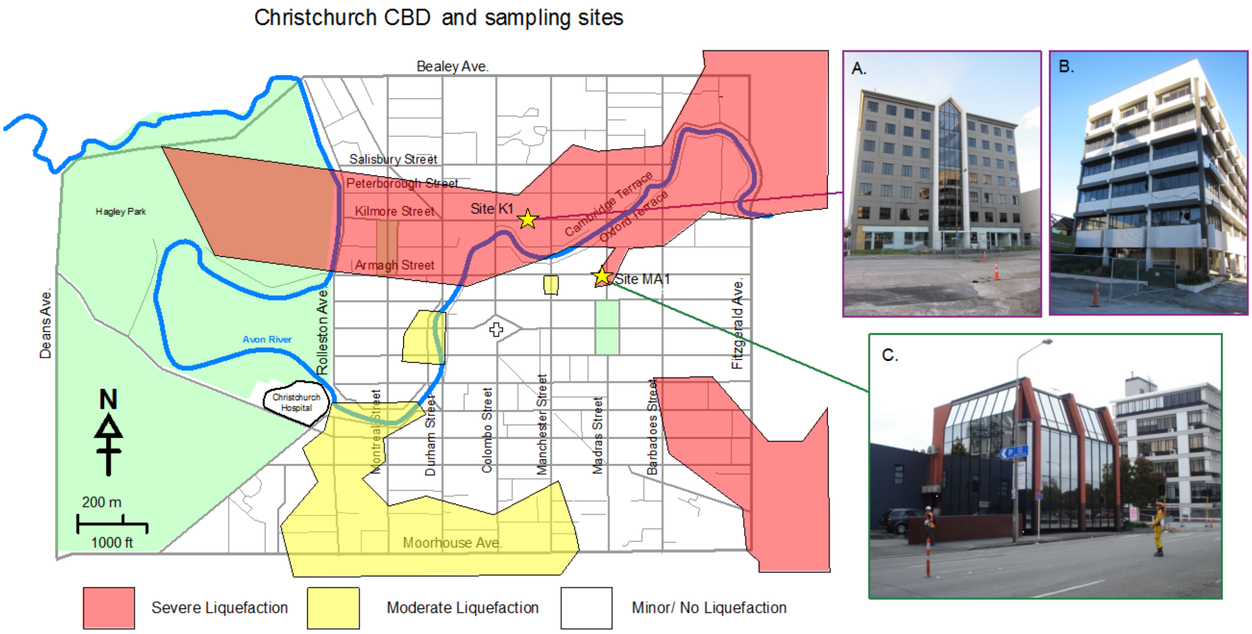


Figure 20: Map of Christchurch CBD showing the location of sampling sites and the observed zones of liquefaction following the 22 February 2011 Christchurch earthquake. Inset A: Transport House, founded on shallow pad foundations. B: Markham's building, founded on piled foundations. C: Amicus House (foreground), and Trade Union Building (background), both on shallow foundations.

5.2.2 Sampling operation

Prior to sampling, the sites were investigated with CPT to identify consistent soil layers suitable for sampling. Figure 21 presents CPT profile information near sample hole K1. The main sampler used was the GP-S type sampler, but the GP-TR was also trialled at sample hole K1 in medium dense sands. The GP-S provided better quality samples and remained the focus for the investigation in the silty sands encountered in the upper 8m of the soil profile.

The drilling and associated field sampling activities were led by Mr. Iain Haycock of McMillan Drilling Ltd., with assistance from experienced geotechnical technicians from Kiso Jiban Consultants of Japan, the developers of gel-push sampler technology. Figure 21 show photos taken from the sampling operation at the MA-1 site.

Problems included encountering gravels down-hole, which on one occasion damaged the cutting shoe blade. It is not known whether the gravel was from the sampled formation or had fallen from the upper-layer of fill. The other problem encountered, but not identified until after the sampling trials were completed, was that the New Zealand obtained PVC inner tube for use with the GP-S sampler had a slightly smaller internal diameter (ID 70.6 mm) than the Japanese tube (ID 71.4 mm) that the sampler had been designed for (note the cutting shoe blade is 71.2 mm dia.). This meant it was possible for sampled soil to be sheared against the internal sample liner with the NZ PVC tube, while a small inside clearance was maintained with the Japanese tube. This affected samples from K1-6 sample tube, and all tubes collected from MA-1 sample hole. The GP-TR trial sampling resulted in either loss of soil down the borehole (no recovery), or samples that exhibited visual damage or distortion (shear cracking, slumping, dilation of the sample within the sample tube). Only two attempts were made to sample at the K1 site, and no trials with GP-TR were made at the MA-1 site.

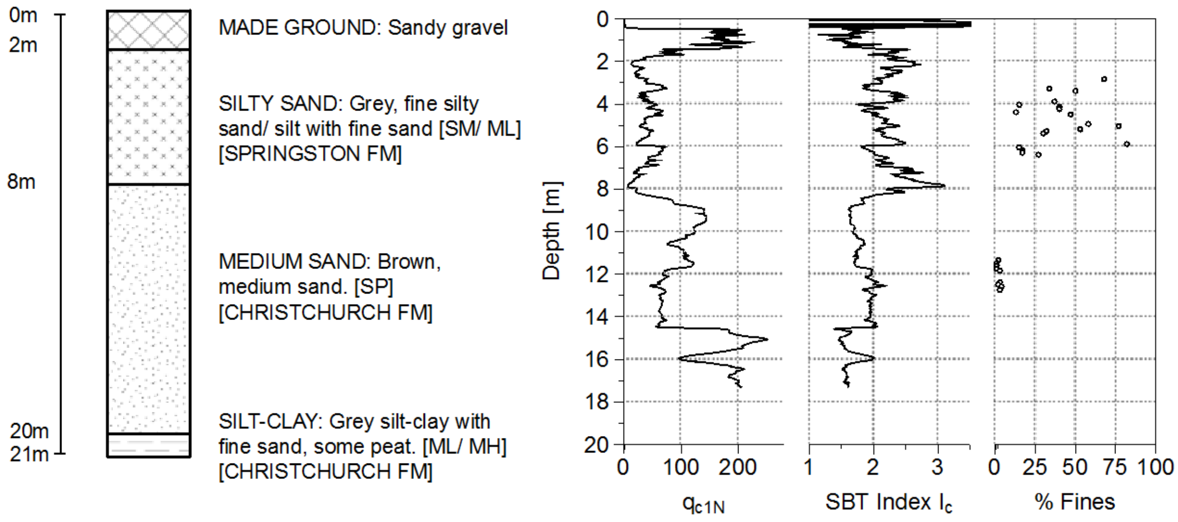


Figure 21: Sampling site K1 geotechnical profile. Left: Summary borehole log and soil unit description, Right: CPT normalised cone resistance, q_{c1N} and soil behaviour type index, I_c, and GP sample fines content.

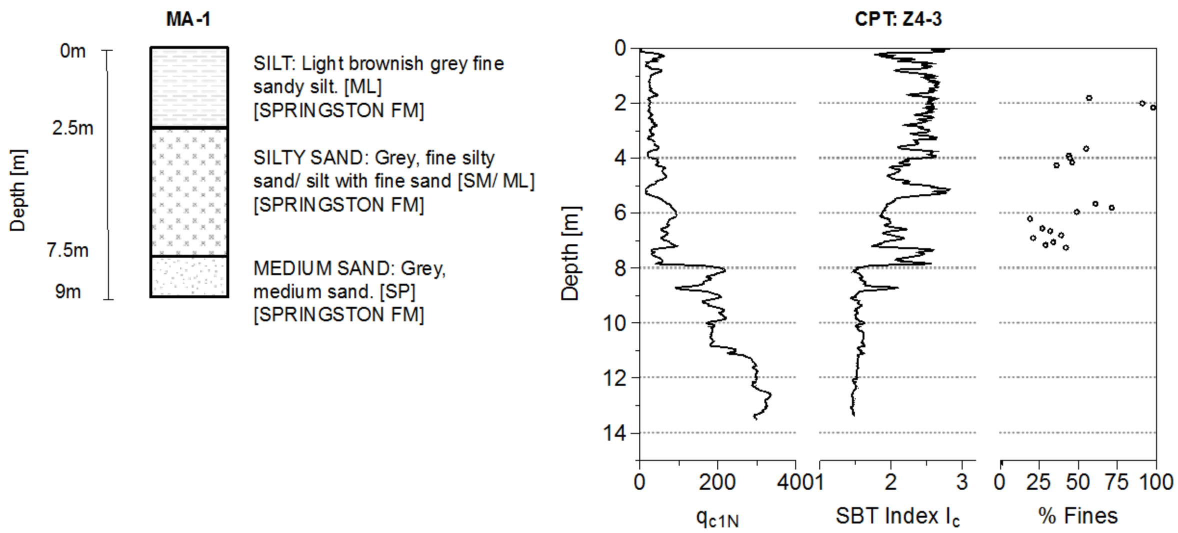


Figure 22: Sampling site MA1 geotechnical profile. Left: Summary borehole log and soil unit description, Right: CPT normalised cone resistance, q_{c1N} and soil behaviour type index, I_c, and GP sample fines content.

Figure 21: Photos from the drilling and sampling operation at Madras-Armagh Street site (MA-1), August 2011 (A, B, D). Removal of sampler from downhole (C), and extraction of internal sample PVC liner with gel-polymer lubricant (G, H, J). The disposable core catcher (E) is shown engaged behind the cutting shoe (F, I).

5.2.3 Materials encountered

Photos of retrieved GP-S samples from both the fluvial silty sands and clean marine sands at the Kilmore site are shown in Figure 24. The silty sand sample shows layering of the silts within or adjacent sandier layers within the sample, while the marine sand exhibits a more uniform structure.

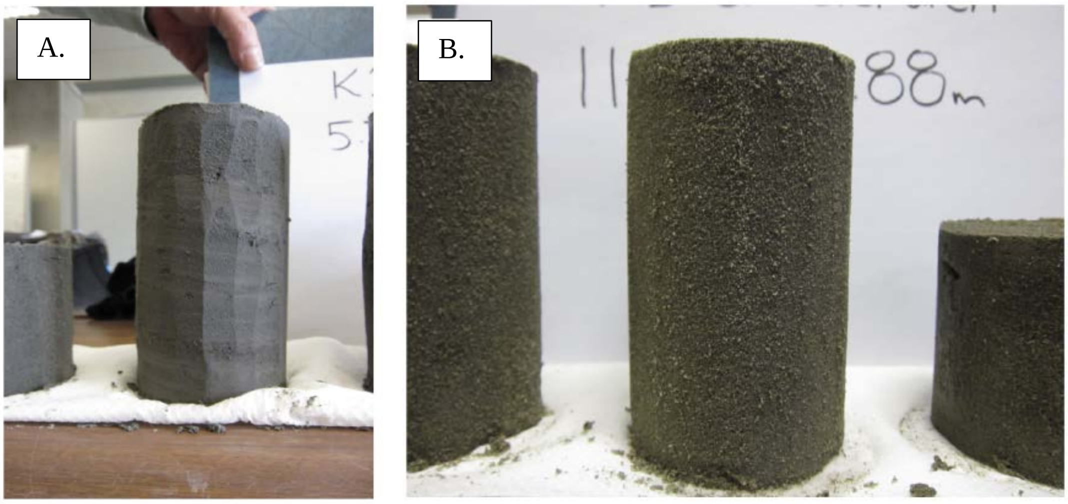


Figure 24: Photographs of GP specimens from borehole K1. A.) Silty sand sample from ~5m depth, outer edge has been trimmed to reveal the layering of the specimen. B.) Clean marine sand from ~11.5m depth.

The soil samples retrieved from boreholes K1 and MA1 varied in soil gradation, notably by fines content (FC, % passing the 75 μ m sieve), while the gradation of individual samples were relatively uniform. Retrieved samples varied from clean fine sands (fines content, FC < 5 %) to silty-sands (FC 12 – 30 %), sandy silts (30 – 50 %) and finally silts (FC > 50 %). Soil profile plots at sampling sites K1 and MA1 in Figure 21 and Figure 22 respectively show the range of sample FC with depth. Particle Size Distribution (PSD) curves for the K1 GP samples are shown in Figure 24, grouped by FC range with separate plots for the Springston and Christchurch Formation soils (fluvial silty sand and marine sand respectively). The clean marine sands have a mean grain size, D_{50} between 0.2 and 0.3 mm, similar to the Japanese benchmark Toyoura sand (0.17 mm), and the Canadian Fraser River sand (0.26 mm) but with less uniformity. The sand is finer than Monterey No. 0 sand (0.38 mm) and Ticino sand (0.58 mm), benchmark sands tested in the US and Italy respectively.

The mineralogy of the sands were considered by Taylor (2015) using X-ray diffraction analysis (XRDA), yielding approximately 65:35 to 70:30 ratio of quartz to albite (sodium feldspar), with trace kaolinite (low activity clay) and muscovite. This is consistent with the siliceous mineralogy of the source rock of greywacke sandstone and argillite siltstone originating from erosion of the Southern Alps, and transportation by rivers to the site or by rivers and ocean currents/ surf (marine sand).

Scanning electron microscope (SEM) imagery of the grains of sand show them to be sub-angular but otherwise moderately spherical, while the silts varied in shape with size, the larger grains being angular to sub-angular moderately spherical grains, with the finer particles angular, elongate and some plate-like.

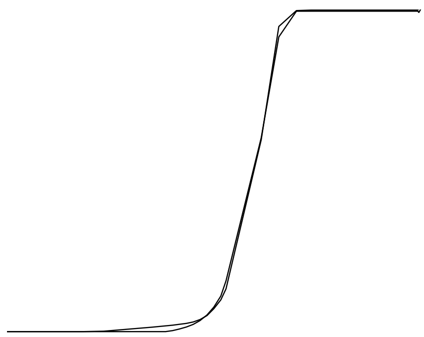


Figure 24: Particle size distribution for all K1 samples grouped by geological formation and fines content. Left: Fluvial silty sands of the Springston Formation. Right: Marine sands of the Christchurch Formation.

5.2.4 Quality of samples retrieved

Quality evaluation procedures considered

The GP samples obtained from the sampling trial were evaluated for sample quality using a range of methods, including both qualitative and quantitative methods. Qualitative methods included visual inspection for signs of disturbance such as cracking and sample distortion, and attempts to measure changes in volume during reconsolidation against published values in the literature. Quantitative methods included:

- Measurement of sample dimensions for comparison to the cutting shoe blade (idealised sample dimension assuming no distortion occurred prior to sampling);
- Theoretical evaluation of the relaxation strains due to loss of confinement as the samples were removed from the *in situ* state to the ground surface;
- Measurement of changes in sample fabric via comparison of field and lab measurements of shear wave velocity, V_s (e.g. Huang et al., 2008; Ishihara, 1996; Kishida et al., 2009; Tatsuoka and Shibuya, 1992; Jamiolkowski et al., 2014);
- Measurement of changes in sample density via field correlations between CPT and relative density, D_R or state parameter ψ, and laboratory measurements of sample void ratio.

Comments on quality evaluation procedures

Of these methods the most valuable for determining the likely sample quality were considered to be visual inspection of sample damage and comparisons of shear wave velocity. Visual inspection provides clear signs of disturbance, while measurement of V_s provides a quantitative evaluation of the soil stiffness, which is a function of both density and soil fabric. Visual quality may appear to be high, but if the measured V_s of the sample is significantly different to the field measurement, it suggests a either a significant change in soil density, or a loss of important ageing related fabric effects has occurred due to sampling disturbance. However, direct field to lab V_s comparison is made complex by the lack of precision of field-based measurements, both in terms of resolution (0.5 - 1m typical spacing for downhole measurement), and uncertainties associated with the interpretation of V_s using conventional techniques. The combination of fine-discretisation provided by CPT testing, and direct measurement using V_s at discrete intervals is considered the best combination in order to establish site-specific correlations for comparison of V_s to GP samples at discrete depths. The ideal tool perhaps for field to lab comparisons is the seismic CPT (SCPT), or alternatively the seismic dilatometer (SDMT), coupled with bender element testing in the laboratory.

In undertaking the comparison between field and lab measurements of V_s for the GP samples, significant effort was made to interpret the field V_s profile at the sampling site using the available measurements from the field (surface wave based MASW and SASW testing, and downhole V_s performed in the sampling hole). The quality of these measurements was not considered to be very high, with significant variations between them. The use of the Christchurch-specific CPT-V_s correlation of McGann et al. (2014) based on SCPT data from across Christchurch (post-quake) was useful to compare to the field V_s measurements to determine applicability of the correlation and the measurement data, and where potential problems with individual field measurements may occur. Other published CPT-V_s correlations from the literature were similarly also considered (specifically Robertson, 2009b and Andrus et al., 2009). From these comparisons, the most reliable CPT-V_s correlations were the Christchurch-specific McGann et al. (hereafter M-CPT), and the Andrus et al. correlation (hereafter A-CPT), where the latter has a factor for the age of the soil since deposition or significant disturbance. This correlation matches the field measurements and M-CPT better when the age is set to 1 year, or the approximate time since the most recent major earthquake in the Canterbury Earthquake Sequence, rather than the estimated age of the soil (100’s to 1,000’s of years). Both the generic Robertson correlation and the Andrus et al. correlation (when age factor set to the expected time since first deposition) estimate notably higher V_s than measured (for the Robertson example refer Figure 25 lower plot CPT data compared to downhole V_s). These comparisons suggest a loss of age-related fabric effects from the soil due to the recent earthquakes.

Assessed quality of GP samples

The GP samples at the K1 borehole site were observed to exhibit higher quality than the samples retrieved from borehole MA1. Significant shearing and distortion of the samples from MA1 were observed with few samples considered to be “undisturbed” on the basis of visual inspection alone. This was on account of the problems with the sizing of the internal PVC liners used at the site. The K1 silty sand samples were generally found to be of good quality when compared to the appropriate V_s profile, both from visual inspection and comparisons made based on V_s in both the lab and field. A few individual samples exhibited some visual disturbance, or V_s that did not match as well with the field measurements.

Plots in Figure 25 show the K1 $V_{s,1}$ profile as measured from downhole V_s and using CPT-V_s correlations applied to available nearby CPT traces (red is closest CPT with a band identifying the uncertainty with the CPT-V_s correlation (range mean +/- 1 σ shown), others grey), are plotted alongside the lab V_s data. All V_s data has been normalised for confining stress to 1 atmosphere (subscript 1). The upper-plot shows a reasonably good comparison between the M-CPT profile and the downhole profile (`direct’ interpretation, green line). Bender element (BE) test results on individual GP samples are shown as coloured circles. The colours represent the estimated volumetric strain occurring to the samples post-extrusion, but may not represent the actual volumetric strain induced by the sampling process, which is unknown. Notations indicate samples with observed signs of disturbance. The silty-sand sample data from the upper 8m profile lies within the mean +/- 1 σ range of the M-CPT correlation, and lies about the mean for the clean sands sampled below 11m depth. The sample data lies consistently below the Robertson CPT-V_s profiles (R-CPT), shown in the lower-plot.

Figure 26 presents detailed comparisons of BE data and corresponding field measurement or estimate based on CPT-V_s correlation. Alignment with the red dashed line would indicate parity between lab and field profile. Very good agreement is obtained for the clean sands below 11m depth and downhole and M-CPT V_s profiles (A, B, C, and D, E, F plots). However some bias with depth is observed with both field profiles and the lab data, where the latter is indicating a larger discrepancy for shallow samples, with higher lab measurements of stiffness than the field. This would indicate not a loss of fabric, but possible increase in density due to sampling disturbance for the silty sands. Alternatively, it could be an anomaly of the downhole V_s measurements, due to imprecision of the profile and interpretation of the stiffness profile. An anomaly may also be a feature of the M-CPT correlation since it displays an influence of depth on the correlation. The R-CPT and A-CPT field profiles do not show an influence of confining stress on the correlation by comparison (linear offset with depth in D, E, F and G, H, I plots).

Comparisons with density measurements in the field (CPT-D_R and CPT-ψ correlations) were found to be problematic in terms of accuracy of the correlations. The state parameter correlations rely on the critical state line (CSL) of the samples to be known in order to perform the lab and field comparison, introducing further information required and additional source of uncertainty. However, despite these challenges comparisons were conducted. The state-parameter comparisons generally showed the GP samples at K1 site to be in the right order when compared to the field profile estimates. Silty sands between 2 and 8 m depth in the field were estimated to have a ψ □□between 0 and –0.07, considered to be states that are contractive to mildly dilative, and therefore relatively vulnerable to earthquake induced liquefaction. The clean marine sands below 9m depth exhibited ψ of between -0.07 and -0.15, and as such are considered to be less vulnerable to liquefaction, but may undergo cyclic mobility under strong earthquake shaking. GP samples obtained using the NZ-PVC tube (sample tube 6 at K1 site in Christchurch Formation clean sands), exhibited notably denser states than predicted from the in situ tests, and were generally denser than samples obtained in the same deposit but collected using the correctly sized Japanese PVC tube. This difference in soil state was not measured by the V_s comparisons however, which suggest strong agreement between field and lab for these soils, regardless of sample tube used. This may be due to the relatively low sensitivity of V_s to changes in soil density. Samples of silty sands which exhibit signs of disturbance appear to have denser states than the field would predict (and denser than similar nearby samples).

From the quality evaluations conducted, no single method was found to be entirely satisfactory for measuring sample disturbance in a convincing manner. On this basis, and without a `gold standard' comparison of perfect samples to go by (e.g. frozen or high quality block samples), a meta-quality assessment was conducted whereby an index value (Sample Quality Index, SQI) was given to each sample on the basis of multiple evaluation methods. The qualitative visual evaluation of disturbance, and quantitative V_s lab/field comparison were weighted more than quantitative lab/field comparisons of density and state, and other attempts to measure sample volume change, for a possible total SQI of 10. On this basis 50% of K1 samples were considered to be of good quality (SQI of 6 or more). MA1 samples only identified two samples in this range, the rest being of notably poorer quality.

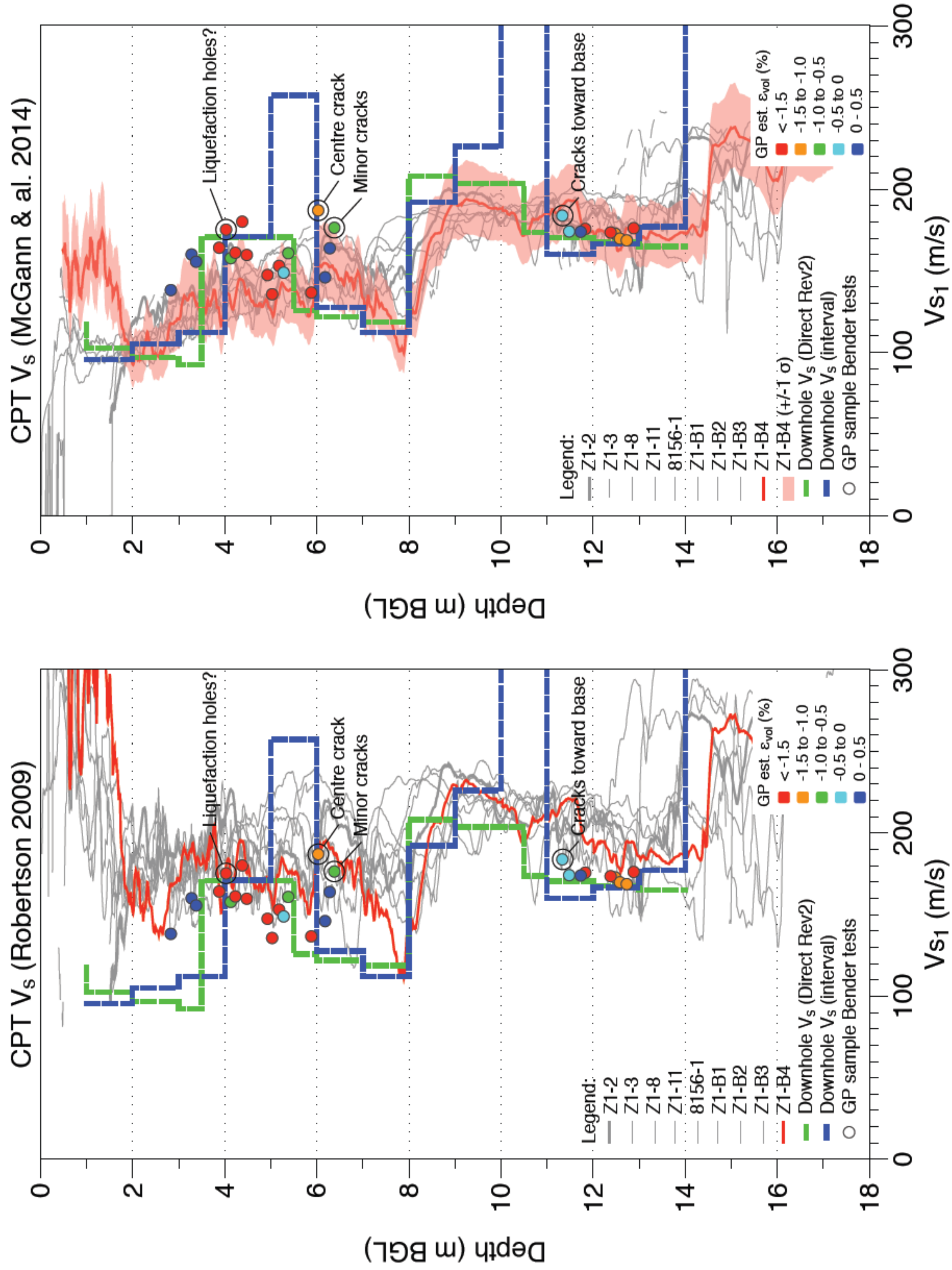


Figure 25: Comparison of K1 GP sample bender element test data with downhole Vs profile, and the estimated profiles from CPT-Vs correlations of Robertson (2009) [bottom plot] and McGann et al. (2014) [top plot]. All data has been normalised for confining stress to 1 atmosphere. The nearest CPT profile (Z1-B4) to the sample hole is shown in red.

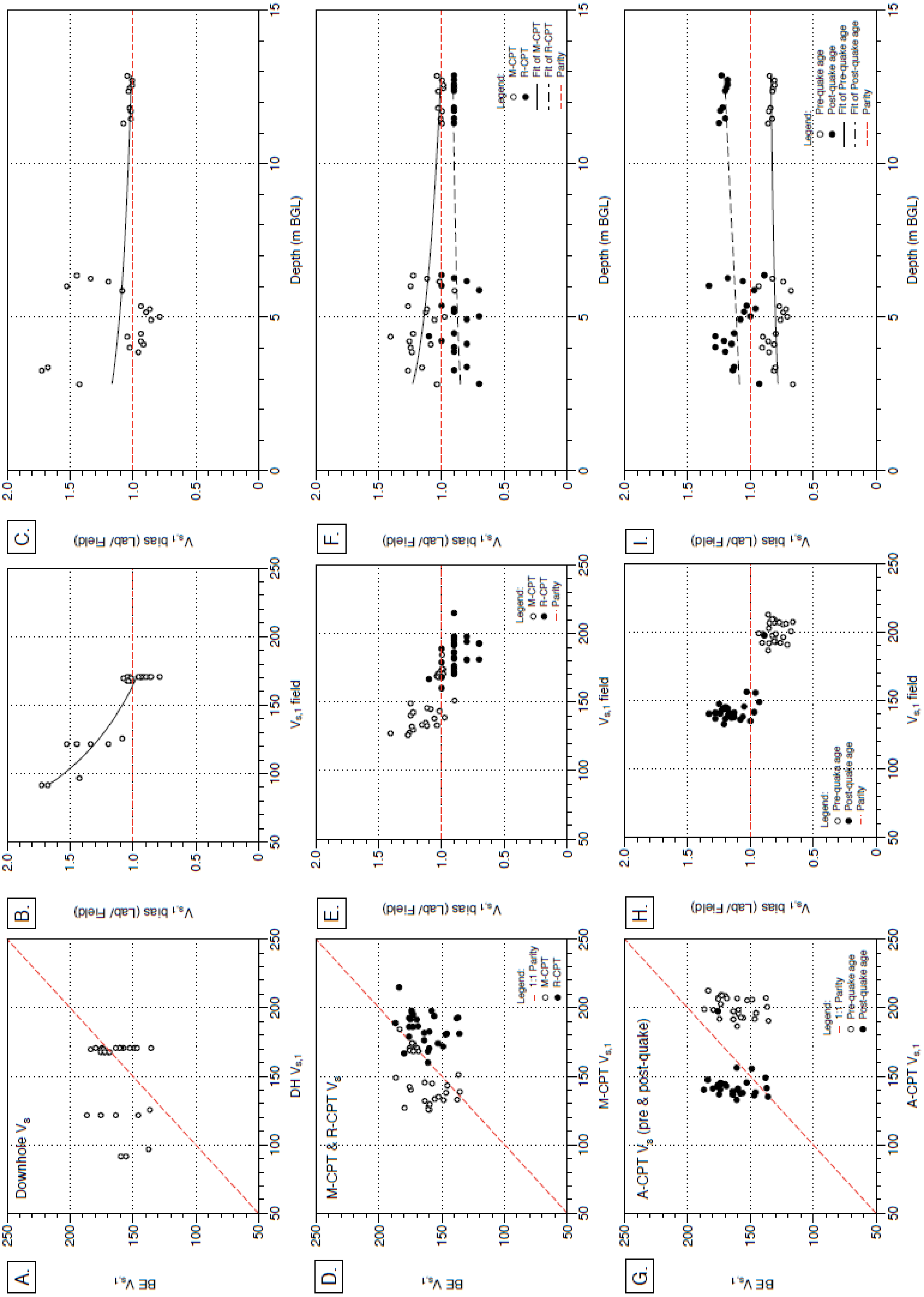


Figure 26: Direct comparison and bias plots between laboratory and field measurements of shear wave velocity using different field $V_{s,1}$ profiles. Plots A, B & C: Plot of $V_{s,1}$ bias with depth. Plots D, E and F present the same for the McGann et al. (2014) & Robertson (2009) CPT-Vs correlations (M-CPT, R-CPT resp.), and G, H, and I for the Andrus et al. (2009) pre-and post-quake age profiles (A-CPT).

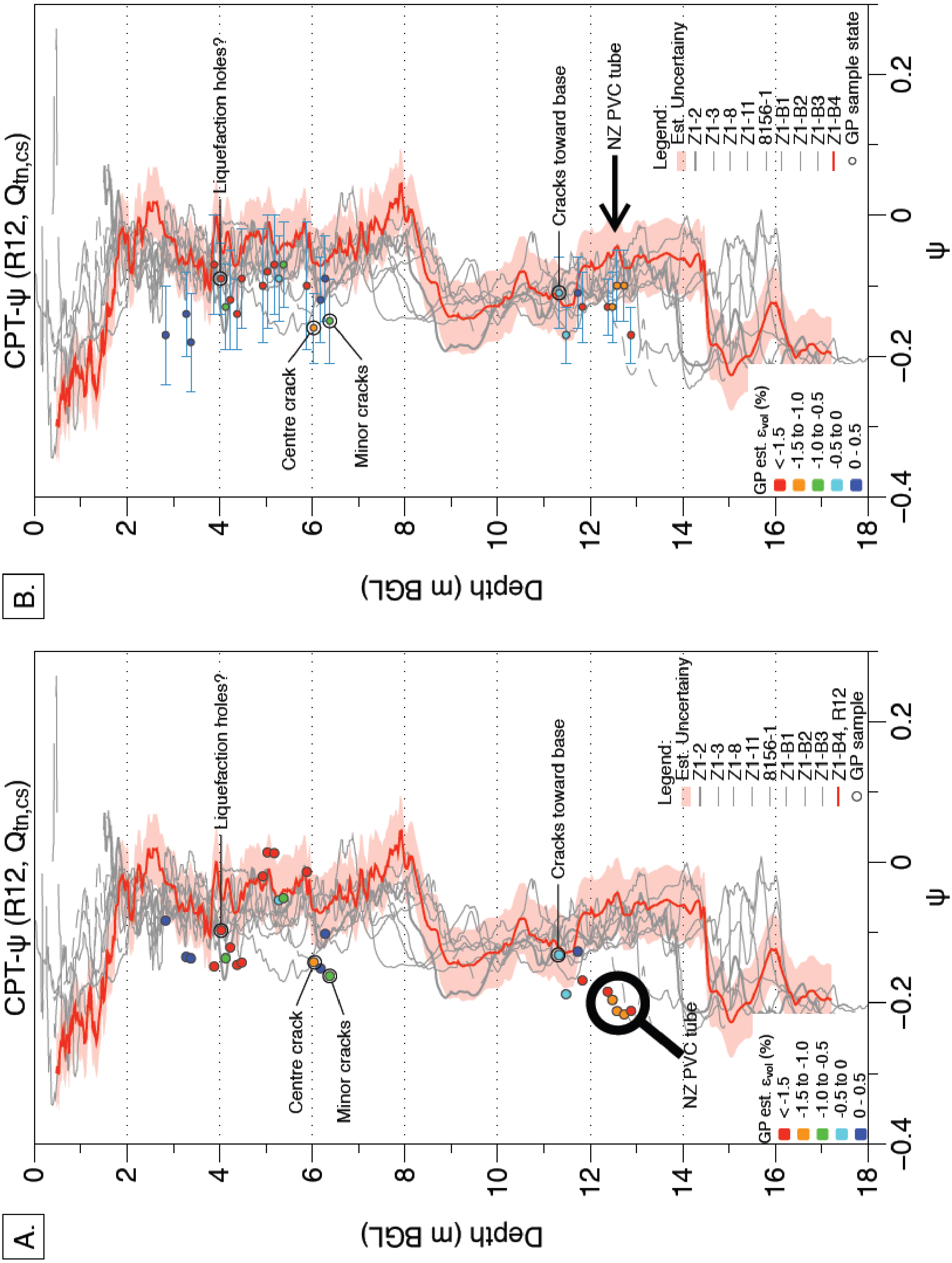


Figure 27: Comparison of field and laboratory estimates of state parameter ψ. CPT-ψ correlation of Robertson (2012) ("R-12"). Lab data considered (A) CSL of nearest `representative soil of similar FC range, while (B) used empirical CSL relationship based on soil index tests (e_{max}, e_{min}), for details refer Taylor (2015).

5.2.5 The Characterisation of Christchurch Sands for Modelling

The GP sampling field trials enabled samples of undisturbed samples of Christchurch silty sands to be collected and tested for in the laboratory for the first time. The testing and characterisation of Christchurch sands drawing on the use of GP samples formed a major component of the research work presented by Taylor (2015). The results of this testing were used to analyse the influence of natural features of soil on the undrained cyclic response (i.e. liquefaction resistance) and to calibrate an advanced constitutive soil model.

The obtained GP samples were tested in both monotonic and cyclic triaxial testing. The primary focus of the tests of the undisturbed samples was cyclic, in order to establish cyclic strength curves, and the highest quality samples based on visual inspection were reserved for these tests. An example cyclic triaxial test result is presented in Figure 28. A single test result may be represented as a point on a plot which defines the cyclic strength of the soil under varying amplitude of loading. In the case of the test presented in this figure, the load amplitude (expressed as cyclic stress ratio, *CSR*) is 0.26, with the corresponding number of cycles, N_c required to generate ‘cyclic liquefaction’ being 6. Figure 29 presents the cyclic strength curve derived from three test results of silty sands obtained from sample tube K1-3, at depths between 4.8 and 5.4 m depth. The CRR_{15} intercept noted on this plot can be compared to the simplified liquefaction triggering evaluation method. Thus, while not entirely straightforward, the two approaches to evaluating liquefaction triggering may be used concurrently and in a comparative way.

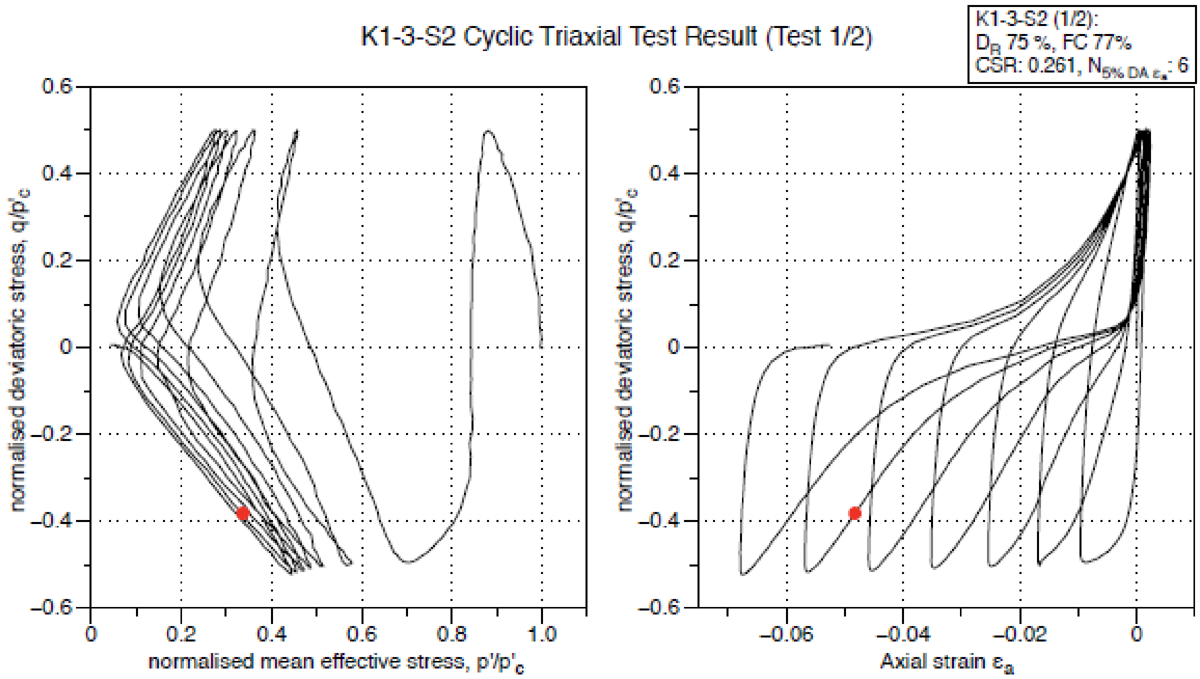


Figure 28: Example cyclic triaxial test result for GP sample K1-3-S2, a silty sand specimen from Kilmore Street sampling site. Left plot: Effective stress path plot during cyclic loading, normalised by initial confining stress p’c. Right plot: Stress-strain plot during cyclic loading. The red dot represents the occurrence of 5 % axial strain in double amplitude (cyclic liquefaction definition).

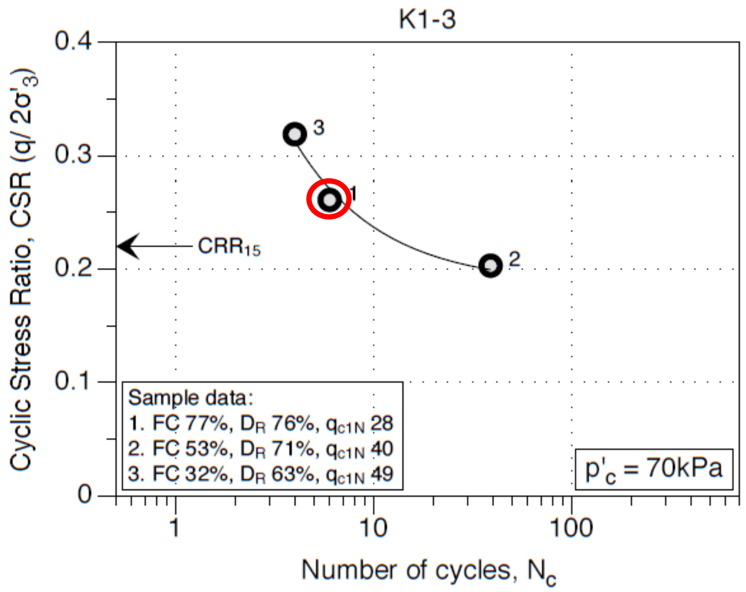


Figure 29: Cyclic strength curve for silty sands from sample tube K1-3. The data point highlighted in the red circle is the test result presented in the previous figure. The CRR_{15} is the intercept at 15 cycles. All samples were tested at an isotropic confining stress, p'_c of 70 kPa.

The remaining samples were tested monotonically, mostly in drained tests (CID), providing indicative stress-strain curves, peak strength, and indication of location of the critical state line (CSL). As the samples were all dense of critical state, the samples tended to dilate and form shear bands during monotonic drained loading, precluding the achievement of `critical state' or constant volume conditions during the test. To identify these state characteristics for the Christchurch sands, further tests on selected reconstituted specimens were conducted to determine the CSL, stress-strain curve, and peak strength information as a function of state. These data are necessary for the calibration of the Stress-Density model (Cubrinovski and Ishihara 1998a, 1998b), which was selected as a framework for the characterisation of these sandy soil materials. For further details on the study and the findings; refer to the Ph.D. thesis of Taylor (2015).

5.2.6 Key findings in relation to GP Sampling from the Initial GP sampler trials

The CBD trials provided good quality samples, but also clearly disturbed samples, introducing us to some of the challenges with the operation of the new samplers in the field. The process of obtaining the samples and testing them in the lab, to assess their sample quality, provided us with an understanding of the key issues with undisturbed sampling and evaluating the reliability of the laboratory test data, in terms of how it reflects the in situ response.

The following recommendations provide directions for future research needs:

- The existing GP-S sampler design appears to be sub-optimal for reducing compression strains on samples as it advances into virgin soil. The width of the sampler results in an overall low B/t ratio, as the sampler must contain an internal liner. As suggested by Lunne and Long (2005), the design of the cutting shoes blade should be lengthened so that the effect of the width of the sampler is minimised during sampling. This would be of benefit in either soft plastic silty clays, or compressible silty sands/ sandy silts. A trade-off exists between reducing friction between the cutting shoe blade and the sandy soil samples, and the need to advance an overall sharp sampler than minimises compression strains on the virgin soil. This may require different cutting shoe lengths for different soil types and densities, requiring an optimisation process, related back to field CPT testing (penetration resistance and interpreted soil type). The existing cutting shoe design may be most suitable for medium dense sands, as from visual inspection and V_s comparisons, some good quality samples were obtained in these soils at the K1 site.

- The use of rotary sampling GP-TR is problematic, and appears to require very dense competent soils to sample, preferably with bonding or cohesive fines, as dense sands are highly sensitive to shearing disturbance. The large diameter of the internal liner allows for expansion of the sample unless the sample retains some cementation or bonding between particles. Further trials and increased proficiency with the drilling and sampling operation are also required.
- Samples need to be transported and stored carefully. Freezing is most practical and beneficial for clean sands, which are more likely to creep and/or distort following extrusion prior to testing. The presence of gel-polymer within the sample structure may prevent successful freezing. In the test programme it is recommended that clean sand samples be tested first, while samples with fines, and consequently higher negative excess pore water pressures helping to retain the sample fabric and shape post extrusion, may follow when the testing on clean sand samples have been completed.
- Sample quality evaluation should be carried out using high quality in situ data to obtain small strain stiffness and in situ void ratio/ state. Non-destructive geophysical techniques are desirable to characterise the V_s and porosity profile. Not all V_s profiling methods have the required accuracy and precision necessary to allow for close comparison to measurements of laboratory specimens, e.g. surface wave based SASW and MASW techniques, which infer a V_s profile with depth. Downhole logging performed at 0.5 or 1m centres is also too coarse a resolution for detailed sample quality comparisons. Suggested methods for in situ void ratio measurement include downhole gamma-density logging (especially the Radioactive Isotope CPT cone, Mimura et al. 1995) e.g. as used on the CANLEX project (Wride et al. 2000b), and downhole/ crosshole V_s and V_p testing to obtain porosity and hence in situ void ratio (Foti et al. 2002, Foti and Lancellotta 2004). In all cases a continuous profile of data is needed to allow accurate comparisons to individual samples at particular depths. For this reason, the use of closely located CPT to the sampling hole is recommended, allowing use of CPT-V_s correlations, validated or corrected to site-specific through direct measurement of V_s (e.g. SCPT, or downhole seismic in the sampling hole). The Measured to Estimated Velocity Ratio (MEVR) approach presented by Andrus et al. (2009) provides a suitable means to correct a correlation to site-specific measurements. The SDMT test presents a potentially useful means to evaluate the field response at small, intermediate and large strains, and may have application to assessing the quality of samples across these ranges in a comprehensive way.
- Further research into the effects of fines on CPT cone resistance and the development of appropriate correlations with soil state (D_R, state parameter) is warranted, such as calibration chamber studies. To date these studies have largely focussed on clean sands. This should also entail revision of procedures to derive e_{max} and e_{min} to appropriately account for the influence of fines.

5.3 The Silty Soils Project

During the assimilation of data following the Canterbury earthquakes of 2010-2011, it was noted that at some sites where liquefaction triggering would have been predicted to be severe using conventional semi-empirical methods (i.e. as used in engineering practice), no liquefaction was observed, and vice-versa. The semi-empirical methods used to estimate liquefaction triggering and subsequent damage are necessarily conservative due the significant scatter in the case-history dataset and the need to avoid false-negative predictions. The data that informs the curve is a database comprising predominantly clean sands and to a lesser extent non-plastic silty sands, that either liquefied or did not liquefy during historic earthquake events. Christchurch typically has a range of fluvial deposits, from clean sands to silty sands, as well as sandy silts and silt-clay soils which may be non-plastic or exhibit some low to moderate plasticity.

While conducting site specific studies on the liquefaction resistance of a particular soil can help to reduce uncertainty in the design of a building's foundations, particularly on soil types that differ from clean quartz sands, the additional costs and lack of testing facilities in New Zealand mean that this is seldom carried out. However, the rebuild efforts in Christchurch have highlighted the costs associated with this additional layer of conservatism, raising the question of whether Christchurch silty soils have a larger than expected liquefaction resistance, and if so, whether this can be reliably and easily predicted using the semi-empirical tools routinely used by practising engineers.

In the following sections, progress from one of the test sites selected for the Silty Soils Project is described and parts have been reproduced from Stringer et al. (2015). As this project continues, similar work will be replicated at a number of additional sites, with the aim of providing guidance to both practicing engineers as well as the wider scientific community regarding the liquefaction resistance of silty soils.

5.3.1 Site selection

In the course of the Silty Soils Project, soil samples will be recovered from a relatively large number of sites in Christchurch, where additional complementary testing will be undertaken to provide the best possible soil characterisation.

An initial selection of approximately 30 candidate sites was carried out based on general observations of predicted and actual liquefaction-induced ground damage during the 2010-2011 Christchurch Earthquakes. In particular, candidate sites targeted areas of the city where existing CPT records suggested the presence of a layer of silty sand between 3 and 8 m below the ground surface. The decision to target these silty soils was driven by the observations that this soil type, commonly in conjunction with highly stratified soil profiles, was often present in areas of the city where the soil performance was much better than expected.

The initial pool of candidate sites was reduced to 6 sites of interest for further testing. The selection of these sites was guided by examination of the CPT records, and consideration of the estimated performance of the site, according to the liquefaction potential estimated according to the simplified method (Idriss and Boulanger, 2008) and other proposed damage indicators which included estimated settlements calculated according to (Zhang et al., 2002), liquefaction potential index (Iwasaki, 1978) and liquefaction severity number (Tonkin and Taylor, 2013).

The three Ground Improvement test sites located in the Red Zone have also been included within this study to complement and take advantage of the detailed characterisation and testing which has already been completed in these regions, as well as providing a set of sites where substantial liquefaction damage both would be predicted and was experienced during the earthquakes.

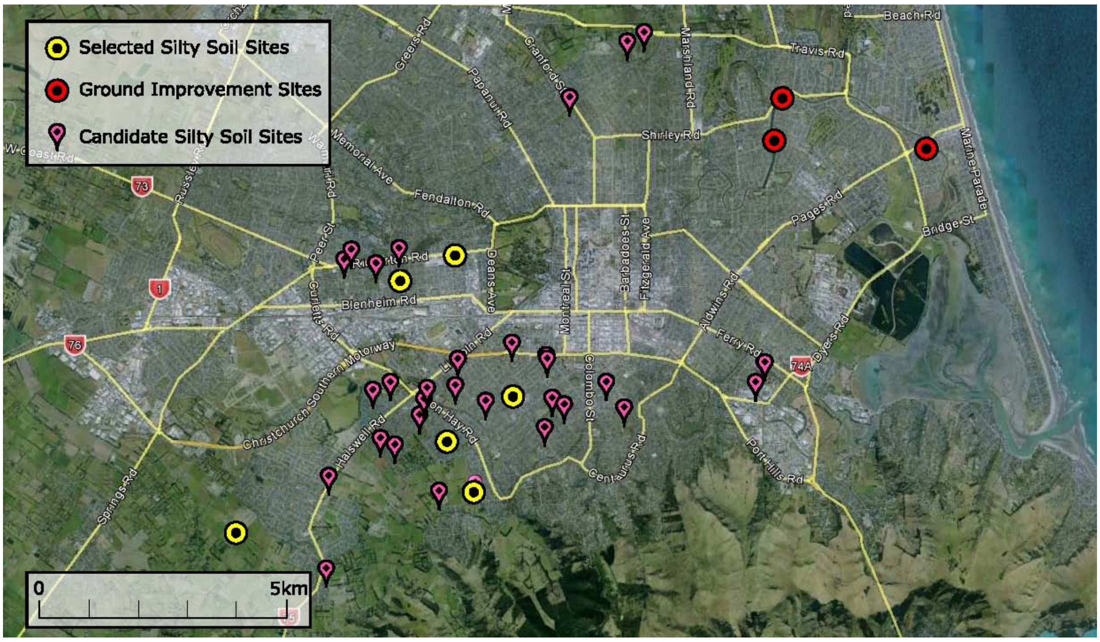


Figure 30: Location of sites for Silty Soils Sampling

An overview of the test results from soil samples obtained at Gainsborough Reserve are presented in the following sections, while detailed results (including individual triaxial test results) can be found in Stringer et al (2015). This site, shown in Figure 31 (a) is located in the Hoon Hay suburb of Christchurch, about 5 km to the south west of the CBD. It will be shown in following sections that the soil profile is relatively well defined into a number of layers, making it ideal for developing initial curves of liquefaction resistance.

At each of the sites identified for further analysis, a number of complementary tests were carried out to provide detailed soil characterisation information. These included a new CPT close to the location of soil sampling, bore hole sampling, dilatometer testing and detailed cross-hole shear wave testing. In addition to these testing activities, both gel-push sampling and Dames and Moore sampling were carried out. Each testing activity requires at least one unique borehole, meaning that care must be taken to ensure that each new testing activity is not adversely affected by previous work. In the case of the undisturbed soil sampling, a 2m distance to existing work was targeted. The final plan layout showing each of the testing activities carried out at Gainsborough reserve is shown in Figure 31 (b).

(a) Location of Gainsborough Reserve **(b) Plan of testing activities (distances in metres)**

Figure 31: Overview of Gainsborough Reserve

5.3.2 Characteristics & targeted sampling profile

In the period following the 2010 – 2011 Christchurch earthquakes, a shallow monitoring well was installed very close to the South-West corner of Gainsborough Reserve. Based on the data collected to date, it is likely that the water table at the time of the February earthquakes is likely to have been between 0.8m and 1.0m below the ground surface. P-wave and S-wave velocities obtained from the cross-hole investigation (CGD, 2015) is shown in Figure 32. P-wave velocities of approximately 1500 ms^{-1} typically indicate fully saturated soil. Based on this interpretation, full saturation is first reached at about 2.2m below the ground surface. However, between 2.6m and 4.4 m, the p-wave velocity clearly reduces, reaching a minimum of 600 ms^{-1} at a depth of 3.5m. This observation is important in the interpretation of the site's response during the February earthquakes, and will be discussed later

The upper 10 m of soil profile at Gainsborough Reserve comprises a number of layers which, according to a CPT evaluation would suggest layers of clayey/organic material ($I_c \approx 3$) sandwiched between silty sand ($I_c \approx 2$). The CPT information for the sampling location is shown in Figure 33. As shown, the soil profile has been split into a number of different layers for the analysis of cyclic resistance. Although a number of layers appear to have similar soil behaviour type indices (such as layers 2, 4 and 6), the results from these different portions of the soil profile will be affected by differences in the particle size gradation, relative density, and fabric. The ranges of these parameters found in the samples within each layer are summarised in Table 4.

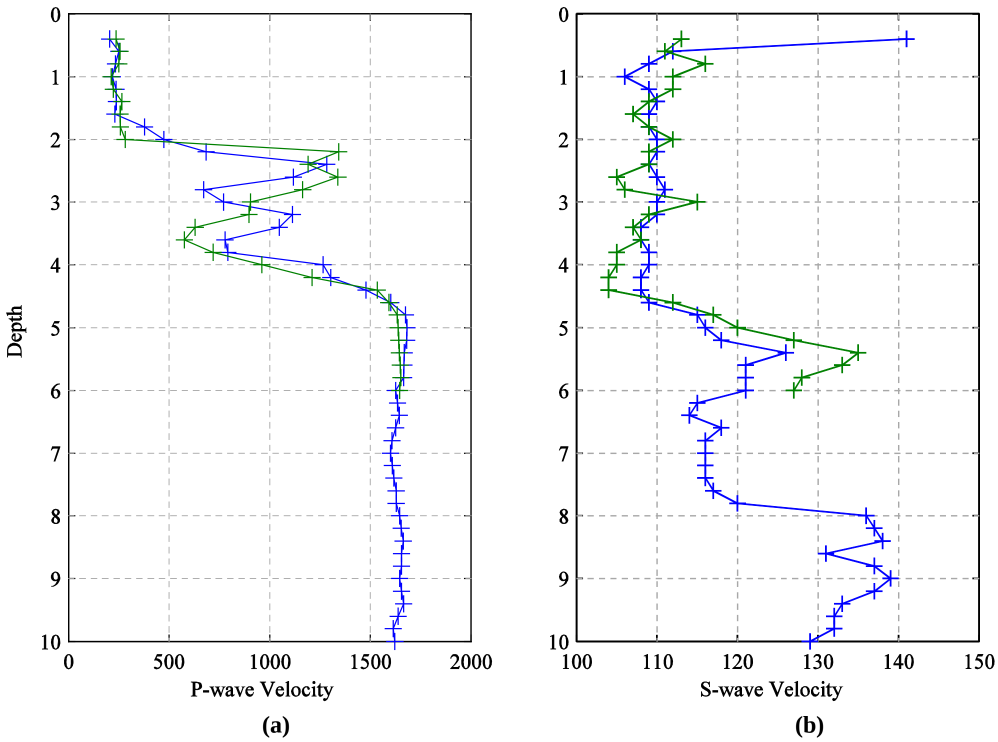


Figure 32: Compression and Shear Wave Velocities at Gainsborough Reserve

The percentage of fine-grained particles (smaller than 75µm) occurring in soil samples is shown in Figure 33(f), while the complete particle size distributions for all samples are shown in Figure 34. It is apparent that even in the layers classified as silty sand ($I_c \approx 2 - 2.6$), the soils are relatively fine grained, but becoming coarser with depth.

Table 4: Sample properties by layer

Layer	Normalised cone resistance q_{c1}	Soil behaviour Type index I_c	Plasticity Index PI (%)	Fines content FC (%)	Median grain size D_{50} (mm)	Post-consolidation void ratio e_c
1	10-29	2.13-2.90	5	63-99	0.012-0.049	0.69-0.87
2	15-35	2.08-2.61	NP-3	77-85	0.025-0.040	0.74-0.80
3	5-6	3.04-3.13	12-17	84-100	0.008-0.013	0.82-1.01
4	38-53	1.84-2.13	NP-10	27-67	0.059-0.106	0.75-0.89
6	38-67	1.85-2.23	NP	9-44	0.082-0.189	0.71-0.79

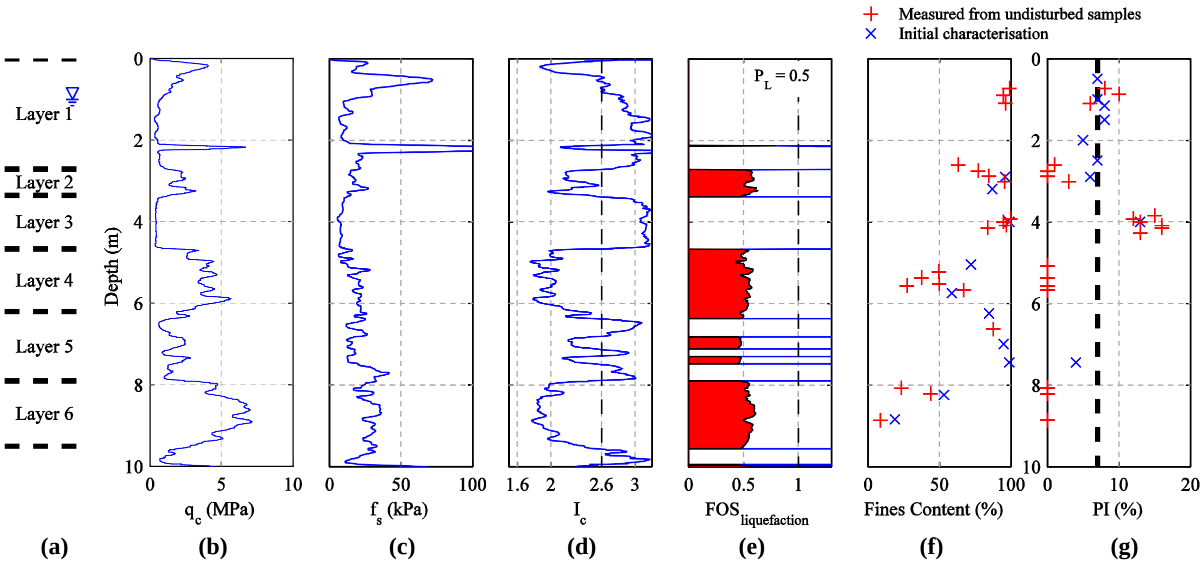


Figure 33: Characteristics of the soil encountered at Gainsborough Reserve

In the simplified method of Idriss and Boulanger (2008) a distinction is made that soils with a PI up to 4 are likely to exhibit "sand-like" liquefaction behaviour, those with a PI greater than 7 (shown as a dashed line in Figure 33(g)) might exhibit "clay-like" cyclic softening behaviour, and "transitional behaviour" observed for soils with PI in the range 4 to 7. The distribution of PI suggests that the soils in Layers 2, 4 and 6 may be susceptible to liquefaction, while the soils in the other layers would be expected to display cyclic softening behaviour.

The simplified method of Boulanger and Idriss (2014) incorporates a probabilistic framework for estimating the factor of safety against liquefaction triggering and for design purposes, the 15th percentile is commonly used to obtain a conservative estimate of liquefaction triggering. Since the actual response of the site is being compared to the predictions, so the curve corresponding to the 50th percentile is shown in Figure 33(e). At this level of probability, the factors of safety against liquefaction triggering range between 0.45 and 0.6, based on the ground motions estimated by O'Rourke et al. (2012).

Despite the simplified methods predicting that liquefaction would be expected at Gainsborough Reserve, the ground surveys conducted immediately after each of the 2010-2011 earthquakes revealed very minor evidence that liquefaction had occurred in the area. This discrepancy could be due to many factors. The demand of the soils in critical layers may not be as high as expected, for example due to early softening of deeper layers. Given that the cyclic resistance has been based on the 50th percentile, the actual resistances of the soils may be higher than that predicted. The silty/clayey layer between the ground surface and the first sandy layer (and interbedded between deeper sandy layers) may be sufficiently robust to prevent surface manifestations from being observed.

Based on the results from the CPT carried out at the site, a number of sampling intervals were defined, targeting soil from each of the soil layers to a depth of 10m. In each soil layer, samples were to be retrieved using both the Dames & Moore and the Gel-Push (GP-S) samplers. Figure 35 shows the depths and soil types (on the basis of the CPT) where sampling was carried out.

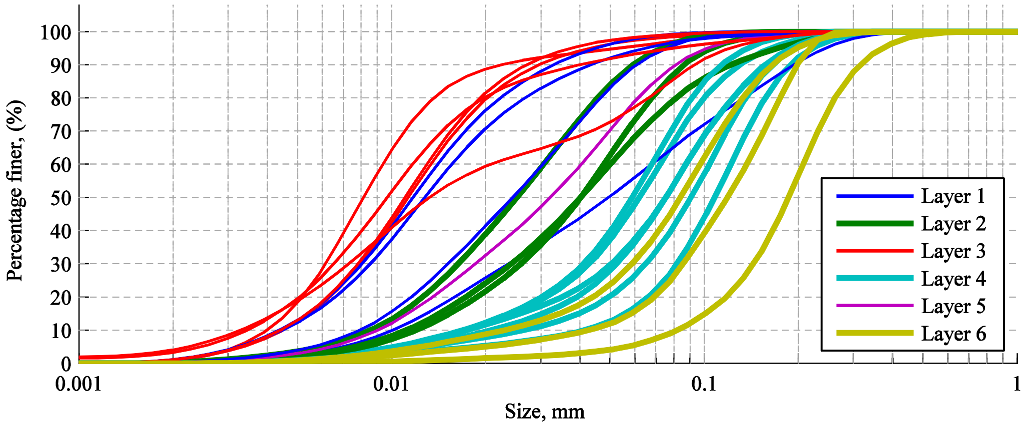


Figure 34: Particle size distributions at Gainsborough Reserve

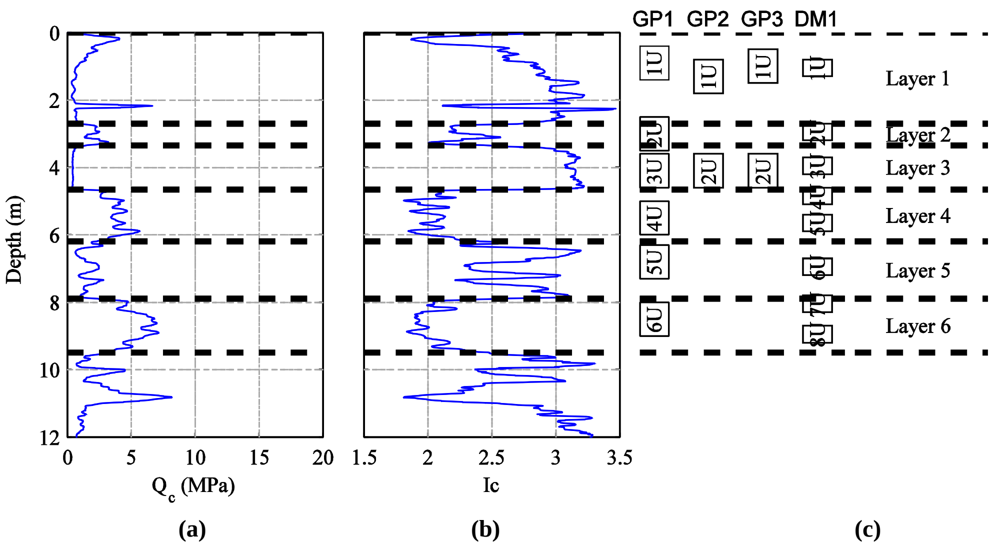


Figure 35: Target sampling profile at Gainsborough Reserve

5.3.3 Sampling Performance at Gainsborough Reserve

Dames & Moore sampling was generally successful at Gainsborough Reserve, with the length of recovered soil greater than 85% of the theoretical maximum for all but one sample. The sample with poor recovery (59%) was taken from a sandier stratum, and the missing portion is assumed to have fallen out when the tool was pulled out of the borehole.

A range in performance of the gel-push sampler was observed at Gainsborough Reserve, with recoveries ranging between 0% and 96% of the theoretical maximum length. In some cases, the low recovery of soil was caused by the core catchers not closing completely, which would have allowed the captured soil to fall out of the tool (since the sidewall friction is eliminated by the gel). It was also found during the trials that a leak had developed in the fixed piston mechanism. The leak can lead to large water pressures acting on the recovered soil specimen, and is likely to have contributed to some of the poor performance of the sampler. Finally a number of specimens exhibited severe cracking and had split into a number of parts. In the preliminary trials of the sampler, the drilling rods were being held in place by the torqueing/clamping arms of the drilling rig. When large hydraulic pressures are being applied to advance the sampler, it was observed that the rods could suddenly slip upwards. This would lift the fixed piston of the sampler, creating a vacuum above the sample and placing the samples in tension. This tensile load is likely to be responsible for the severe cracking and splitting observed on some samples.

To address the known issues with the tool, a number of changes to operating procedures have been introduced, which include conducting a dry run of the tool at the surface prior to the first sampling attempt, and ensuring that the full weight of the drilling rig can be mobilised as a reaction load for the sampler.

In light of the issues experienced in the gel-push sampling, two criteria were used to decide which samples should be used in the testing programme:

- Samples should appear undisturbed (free of major defects) when visually inspected
- The recovered length of the specimen should be greater than 75% of the theoretical maximum.

5.3.4 Liquefaction resistance of Gainsborough Reserve soils

Plots of the effective stress path, stress-strain response, and development of excess pore water pressure with loading cycles from a cyclic triaxial test on a sample from Layer 4 are shown in Figure 36. The soils in Layer 4 are typically fine silty sand, with low plasticity and 60-70% relative density (medium-dense). The limiting minimum and maximum void ratios, measured according to the Japanese geotechnical society standard, were 0.562 and 1.314 respectively. The plots show some of the key features which were observed in most of the tests: strains developed gradually, even after large pore pressures had been generated, and the axial strains which developed were highly biased towards the extensional loading side. Typical of "sandy" soil behaviour, the stiffness of the soil becomes very low during each half cycle, requiring increasing strain each cycle before significant stiffness and strength can be remobilised. The pore pressures measured during this test are shown in Figure 36 (c) as a normalised ratio of excess pore pressure divided by the initial mean confining pressure. In this figure, the points corresponding to zero deviatoric stress are indicated with a red dot.

It was shown in Figure 33 that the soil profile at Gainsborough Reserve consisted of alternating layers of silty sand and clayey/organic material. The response of the latter is distinctly different under cyclic loading on account of moderate plasticity. This is evident in the stress-strain response of a Layer 3 sample shown in Figure 37. The stress-strain loops remain wider, with a gradual cycle-on-cycle reduction in shear stiffness, typical of cyclic softening response.

Using a criteria of 5% peak to peak axial strain, a number of liquefaction resistance curves have been developed for the soils at Gainsborough Reserve, based on the samples retrieved by both the Dames and Moore and gel-push samplers. These curves are plotted in Figure 38 (a) and (b), separated according to whether the soil would be classified as potentially liquefiable using the Idriss and Boulanger (2008) criterion (PI <= 7).

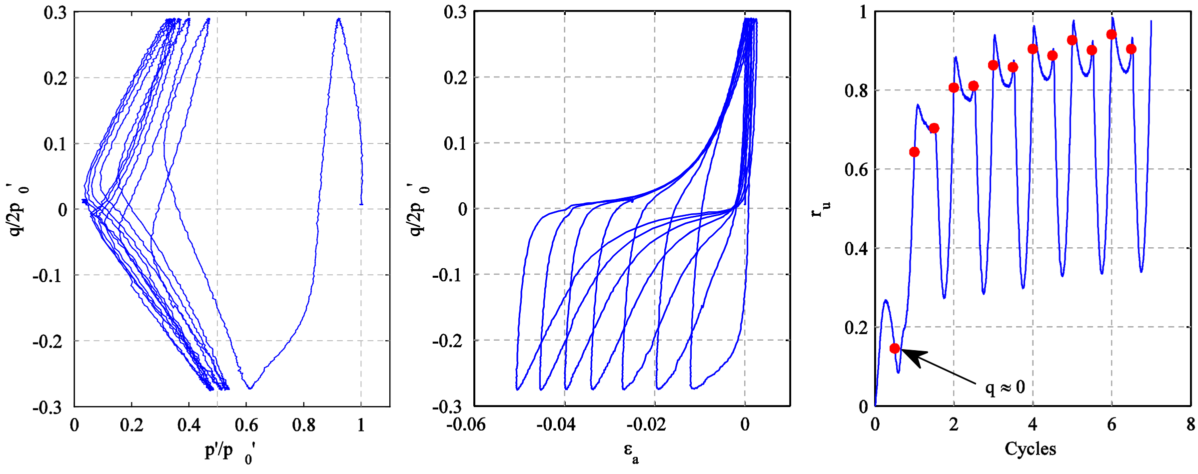


Figure 36: Testing results from soil sample in Layer 4

Cyclic triaxial test results are often fitted with curves following the relationship described in Equation 1, where CRR is the cyclic resistance, *N* the number of cycles of loading and *a* and *b* are curve fitting constants. However, the curves plotted in Figure 38 have been defined in the absence of a mathematical form, instead relying on the data to provide the natural form of the curve. As a consequence, curves have only been drawn for cases where a minimum of 3 data points are available.

$$CRR = aN^{-b} \tag{1}$$

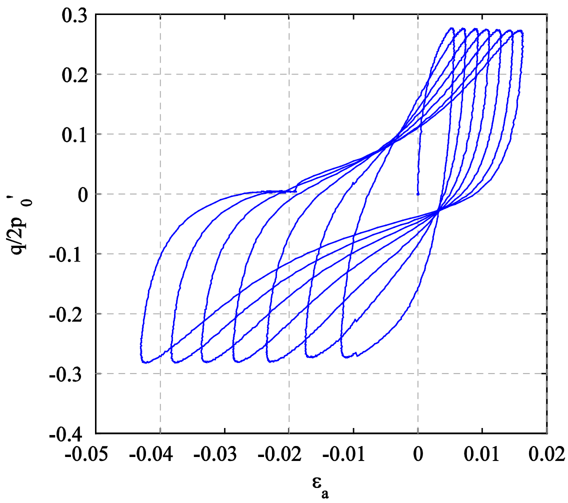
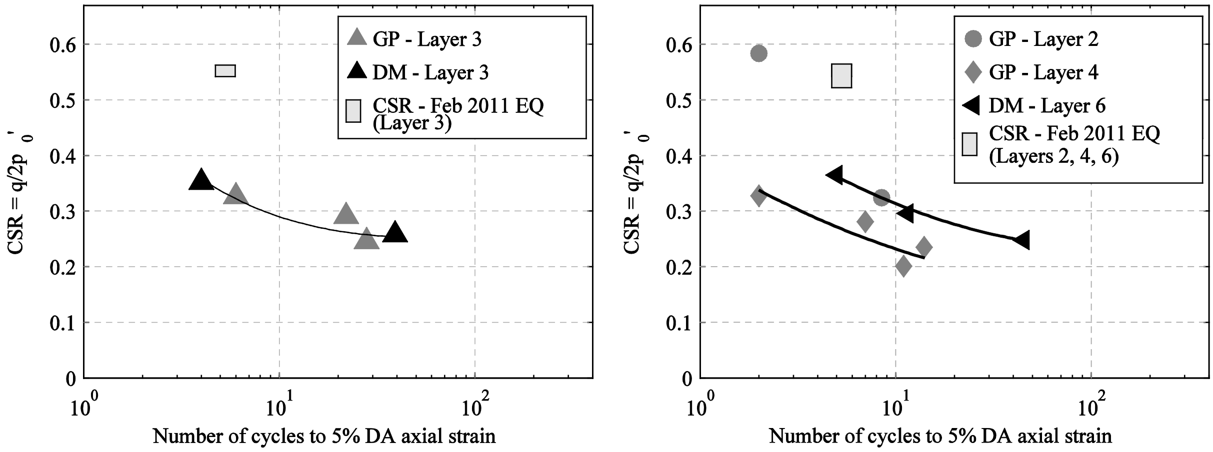


Figure 37: Testing results from soil sample in Layer 3

Examination of the data presented in Figure 38 reveals a number of interesting features. In Layer 3 the individual values of I_c, normalised cone resistance (q_{c1}), PI, FC and average grain size (D50) are very similar so direct comparisons of sampler performance can be made. The data from this layer is shown in Figure 38(a), and suggests that the points from the GP and DM samplers would fall on coincident curves of cyclic resistance. Further, as shown earlier in Figure 13, the shear wave velocity measured in the laboratory for specimens obtained by Gel-Push compares favourably with those obtained by cross-hole measurements in the field. This suggests that the specimens from this layer can be considered of good quality or "undisturbed" for both samplers.



(a) High PI soil (Cyclic Softening) **(b) Low PI soil (Liquefaction)**

Figure 38: Cyclic Resistance of soil at Gainsborough Reserve

Similar comparisons of the cyclic resistance relationships from the sandier soils obtained with the different samplers were anticipated. However, the difficulties encountered during both the sampling and the testing programme meant that cyclic triaxial tests on comparable silty sand specimens (i.e. from similar depths in the soil profile) are not available for this site.

While insufficient data points existed from Layer 2 to define its own CRR curve, it appears that the relationship would be significantly steeper (or potentially higher) than those from Layers 4 and 6, and that the resistance is much higher at low numbers of cycles. Additional scrutiny revealed that the loops of cyclic mobility in a q-p' plot (noting that pore pressure measurements were poor in these tests) showed that the shape of the effective stress path and residual effective stresses that are not typical for liquefiable sandy soils. While the q-ε_a plots showed similar degradation of stiffness at low deviatoric stress, the previous two observations point away from these soils displaying classic liquefaction behaviour. It is possible that the interbedding of the soils within this layer, and the finer grained nature of the soil may play an important role in the response, with any "liquefaction" being constrained to very thin lenses within the layer. Additionally, the saturation in this layer was noted to drop below 100%. The effect of the partial saturation on these silty soils is not well quantified, though studies on clean sands (e.g. Tsukamoto et al. 2002) have indicated that the cyclic resistance would be expected to increase in these circumstances.

The shaded boxes in Figure 38 indicate the estimated demand during the February 2011 earthquake, corrected for the isotropic stress conditions and 1-D loading during the triaxial testing. While the estimated demands are much greater than the strengths of the soils in Layers 3, 4 and 6, it is likely that the cyclic resistance of Layer 2 is close to the applied demands, especially if the effects of partial saturation are considered.

The lack of visible expressions of liquefaction at Site 2 may be explained by a number of different interpretations. The first is that the cyclic strength of the soils in Layer 2 was exceeded during the February 2011 earthquake, but the behaviour of these soils was closer to softening than "liquefaction". Alternatively the increased cyclic resistance of the soil in Layer 2 as a result of partial saturation was sufficient to resist development of 'damaging' liquefaction. Finally, even though liquefaction may have been triggered in the deeper layers (Layer 4), the non-liquefied crust in the top 4.5 m would have supressed liquefaction manifestation on the ground surface. Additional effects may have arisen from the interaction between different layers in the seismic response, including reduced shear stresses due to deeper liquefaction or softening, which may have reduced further generation of excess pore water pressures.

In each of these interpretations, the “critical layer” for surface manifestations of liquefaction becomes Layer 4, which is approximately 4.5m below the ground surface. While it is expected that liquefaction would be triggered in this layer (or a deeper one), it is likely that the thickness of overlying soil was sufficient to prevent liquefied soil from reaching the surface at Gainsborough Reserve. It may also be the case that a mixture of these interpretations was responsible for the better than anticipated site response based on simplified triggering analysis, and the authors aim to confirm these hypotheses in the future through effective stress analyses.

5.3.5 Key Findings

The field work on the silty soils project remains an ongoing research effort which aims to provide greater insights into the liquefaction resistance of the silty soils around Christchurch. While it remains too early to make generalised conclusions, some interesting results have arisen from the advanced sampling and complementary testing carried out at Gainsborough Reserve.

- In the high I_c material (clayey silts), very similar cyclic resistance relationships were obtained for both the Dames & Moore and Gel-Push (GP-S) samplers.
- The results from the laboratory testing suggest that the deeper sandier layers at Gainsborough Reserve would be expected to liquefy under the loading conditions during the February 2011 Earthquake.
- The layer of silty sand between 2 and 3 m below the ground surface was found to be significantly finer grained than expected. Despite expectations of being liquefiable, the behaviour of the soil from this layer was better described by an extreme softening, with a cyclic resistance which was close to the loading estimated during the February 2011 earthquake.
- The cyclic resistance of the shallowest potentially liquefiable layer (between 2 and 3m below the ground surface) is expected to be larger than the laboratory results suggest due to partial saturation of the layer.
- The depth to the first of the deeper sandier layers is likely to have been sufficient to prevent liquefied soil from reaching the surface at Gainsborough reserve, and the apparent discrepancy between the simplified method and the post-earthquake observations.

6. Summary and Conclusions

A number of research programmes at the University of Canterbury aim to establish the in-situ liquefaction resistance of soils found around Christchurch through laboratory testing of undisturbed soil specimens, which retain the effects of fabric, structure and age of naturally occurring soil.

A review of existing sampling methods reveals that a high degree of disturbance is likely to occur if the silty sands found around Christchurch are sampled using normal techniques. The University of Canterbury has acquired a suite of three "gel-push" samplers which were developed in Japan to target similar soils and offers the potential to obtain high quality silty soil samples. To date, two of the samplers (GP-S and GP-TR) have been used in Christchurch. In addition to the gel-push sampling, a *Dames and Moore* sampler has been trialled in Christchurch in collaboration with the University of Berkeley.

All of the sampling trials have been conducted by Christchurch-based McMillan Drilling, with supervision by the original developers (Kiso Jiban Consultants, Japan) during the initial trials in the Christchurch CBD and later with researchers at the University of Canterbury.

Correct handling of soil samples in the field, during transportation and in the laboratory is critical to ensuring that soil samples obtained by advanced methods remain "undisturbed" at the time of testing. In the field, samples should be stored upright, and if the soil is relatively permeable, they should be allowed to drain prior to transportation. Vibration during transportation must be minimised and it is recommended that samples are transported vertically, with padding placed below and around the tubes to minimise the effects of vibration during transport.

The gel-push trials highlighted a number of operational problems, and key lessons for operating the gel-push samplers include the need to carry out surface testing of the tool prior to attempting the first sample and ensuring that a large reaction force can be mobilised to support the drilling rods during sampling.

On the basis of shear wave velocity comparisons with the field, the results from the laboratory testing programmes have so far indicated that the GP-S sampler has been successful in obtaining high quality samples of silty sand and low plasticity silts.

The attempts to sample clean sands using the GP-TR sampler have not been successful to date, showing a marked drop in normalised shear wave velocity when compared against the values measured in the field. Some changes to the operating procedures are planned for future trials of the device.

Direct comparison of the cyclic resistance of the Dames and Moore sampler and the GP-S sampler in a low plasticity silt revealed very similar relationships.

The range of soils sampled using both the gel-push and *Dames and Moore* samplers remains relatively small. Ongoing sampling and testing activities aim to define the envelope of soil characteristics where these samplers can be used with confidence to obtain undisturbed samples.

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Appendix A

A1: Sample tube geometry indices.

Commonly cited sample tube geometry indices include: Inside clearance ratio (ICR); Outside cutting edge angle (OCA); Inside cutting edge angle (ICA); thickness (t), and area ratio (AR). Similar index to AR is the B/t ratio, where B is the outside diameter of the sample tube. Another sampler geometry index is the length to diameter ratio (L/D). These geometries are defined as follows, with reference to Figure 27:

$$ICR = (D_i - D_c)/D_c \qquad [1]$$

$$OCA = tan^{-1} \frac{D_e - D_c}{2 \cdot H_2} \qquad [2]$$

$$ICA = tan^{-1} \frac{D_i - D_c}{2 \cdot H_1} \qquad [3]$$

$$AR = \frac{D_e^2 - D_c^2}{D_c^2} \qquad [4]$$

Figure 27: Sampler cutting shoe/ sample tube cutting edge geometry, after Clayton and Siddique (1999).

Area Ratio and Cutting Edge Taper Angle:

NGI recommendations (Andresen, 1981; Broms, 1980; Lunne and Long, 2005) for the sampling of soft cohesive soils follow. Better samples are retrieved with lower area ratios, and cutting edge taper angles, refer Table 3 for ISSMFE guidance.

Table 5: ISSMFE recommended combinations of area ratio and cutting edge taper. After Broms (1980).

Area Ratio (AR) (%)	Cutting edge taper angle (º)
5	15
10	12
20	9
40	5
80	4

For long seabed samplers featuring a liner, Lunne and Long (2005) recommend the following:

AR < 17%; *t*, of thin walled steel blade of between 3.5 and 5mm, at the cutting head; OCA as sharp as practicable, but certainly less than 10º, with 5º being recommended for soils most susceptible to disturbance; and 0 < ICR < 0.5%.

Length to diameter ratio and inside clearance:

ICR and L/D ratio affect the degree of disturbance to samples due to wall adhesion and friction. Reducing the L/D ratio helps to control jamming of samples in tubes (plug formation). ISSMFE (1981) recommendations for maximum L/D ratios for sampling in different soils is presented in Table 6.

Table 6: ISSMFE recommended length to diameter ratio by soil type. After Clayton et al. (1995).

Type of soil	Greatest Length to Diameter Ratio (L/D)
Clay (sensitivity > 30)	20
Clay (sensitivity 5 – 30)	12
Clay (sensitivity < 5)	10
Loose frictional soil	12
Medium loose (sic) frictional soil	6